NCHRP Project 17-87

Enhancing Pedestrian Volume Estimation and Developing HCM Pedestrian Methodologies for Safe and Sustainable Communities

Proposed HCM Chapters

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Chapter 18: Urban Street Segments

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This chapter contains revisions to Steps 8 and 9 of the urban street pedestrian methodology (Section 4) related to calculating and applying the roadway crossing difficulty factor. These revisions were developed through NCHRP Project 17-87.

CHAPTER 18
URBAN STREET SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

This chapter describes methodologies for evaluating the operation of each of the following urban street travel modes: motorized vehicle, pedestrian, bicycle, and transit. Each methodology is used to evaluate the quality of service provided to road users traveling along an urban street segment. A detailed description of each travel mode is provided in Chapter 2, Applications.

The methodologies are much more than just a means of evaluating quality of service. They include an array of performance measures that fully describe segment operation. These measures serve as clues in identifying operational issues and provide insight into the development of effective improvement strategies. The analyst is encouraged to consider the full range of measures associated with each methodology.

This chapter describes methodologies for evaluating urban street segment performance from the perspective of motorists, pedestrians, bicyclists, and transit riders. The methodologies are referred to as the motorized vehicle methodology, the pedestrian methodology, the bicycle methodology, and the transit methodology. Collectively, the methodologies can be used to evaluate an urban street segment operation from a multimodal perspective.

Each methodology in this chapter is focused on the evaluation of a street segment (with consideration given to the intersections that bound it). The aggregation of segment performance measures to obtain an estimate of facility performance is described in Chapter 16, Urban Street Facilities. Methodologies for evaluating the intersections along the urban street are described in Chapters 19 to 23.

A street segment'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 motorized vehicle travel speed, motorized vehicle stop rate, automobile traveler perception score, pedestrian travel speed, pedestrian space, pedestrian level-of-service (LOS) score, bicycle travel speed, bicycle LOS score, transit vehicle travel speed, transit wait–ride score, and transit passenger LOS score.

The four methodologies described in this chapter are based largely on the products of two National Cooperative Highway Research Program (NCHRP) projects (*1*, *2*). Contributions from other research are referenced in the relevant sections.

CHAPTER ORGANIZATION

Section 2 of this chapter presents concepts used to describe urban street operation. It includes guidance for establishing the segment analysis boundaries and the analysis period duration and describes how an urban street segment is defined for the purpose of this chapter. It concludes with a discussion of the service measures and LOS thresholds used in the methodology.

Section 3 presents the methodology for evaluating motorized vehicle service along an urban street segment. It includes a description of the scope of the methodology and its required input data. It concludes with a description of the computational steps that are followed for each application of the methodology.

Section 4 presents the methodology for evaluating pedestrian service along an urban street segment. It includes a discussion of methodology scope, input data, and computational steps.

Section 5 presents the methodology for evaluating bicycle service along an urban street segment. It includes a discussion of methodology scope, input data, and computational steps.

Section 6 presents the methodology for evaluating transit rider service along an urban street segment. It includes a discussion of methodology scope, input data, and computational steps.

Section 7 presents guidance on using the results of the segment evaluation. It includes example results from each methodology and a discussion of situations in which alternative evaluation tools may be appropriate.

RELATED HCM CONTENT

Other *Highway Capacity Manual* (HCM) content related to this chapter includes the following:

- Chapter 16, Urban Street Facilities, which describes concepts and methodologies for the evaluation of an urban street facility;
- Chapter 17, Urban Street Reliability and ATDM, which provides a methodology for evaluating travel time reliability and guidance for using this methodology to evaluate alternative active traffic and demand management (ATDM) strategies;
- Chapter 19, Signalized Intersections, which provides methods for evaluating pedestrian and bicycle LOS at intersections, the results of which are used in this chapter’s facility-level pedestrian and bicycle methods;
- Chapter 29, Urban Street Facilities: Supplemental, which provides details of the reliability methodology, a procedure for sustained spillback analysis, information about the use of alternative evaluation tools, and example problems demonstrating both the urban street facility methodologies and the reliability methodology;
- Chapter 30, Urban Street Segments: Supplemental, which describes procedures for predicting platoon flow, spillback, and delay due to turns from the major street; a planning-level analysis application; and example problems demonstrating the urban street segment methodologies;
- Chapter 31, Signalized Intersections: Supplemental, which describes procedures for predicting actuated phase duration; lane volume distribution; saturation flow adjustment factors for pedestrian, bicycle, and work zone presence; and queue length; and presents a planning-level

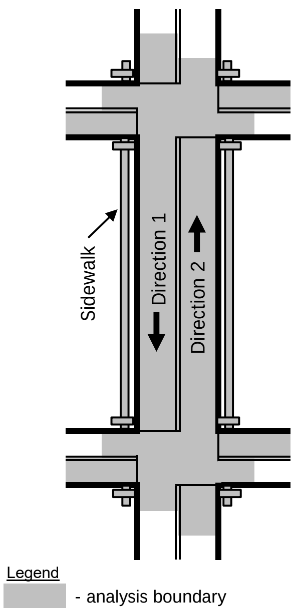
analysis application, as well as example problems demonstrating the signalized intersection methodologies;

- Case Study 3, Krome Avenue, in the HCM Applications Guide in Volume 4, which demonstrates the application of HCM methods to the evaluation of a real-world urban street; and
- Section K, Urban Streets, in Part 2 of the *Planning and Preliminary Engineering Applications Guide to the HCM*, which describes how to incorporate this chapter's methods and performance measures into a planning or preliminary engineering effort.

A procedure for determining free-flow speed when a work zone is present along the segment is provided in the final report for NCHRP Project 03-107, Work Zone Capacity Methods for the HCM. This report is in the Technical Reference Library in online Volume 4.

Methodologies for quantifying the performance of a downstream boundary intersection are described in Chapters 19 to 23.

Spatial and Temporal Limits



For the motorized vehicle methodology, a segment evaluation considers both directions of travel (when the street serves two-way traffic).

2. CONCEPTS

This section presents concepts used to describe urban street operation. The first subsection assists the analyst in determining the type of analysis to be conducted and includes guidance for establishing the segment analysis boundaries and the analysis period duration. The second describes how an urban street segment is defined in terms of points and links. The third discusses the service measures and LOS thresholds used in the methodology. The last identifies the scope of the collective set of methodologies.

ANALYSIS TYPE

The phrase *analysis type* is used to describe the purpose for which a methodology is used. Each purpose is associated with a different level of detail, since it relates to the precision of the input data, the number of default values used, and the desired accuracy of the results. Three analysis types are recognized in this chapter:

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

These analysis types are discussed in more detail in Chapter 2, Applications.

Analysis Boundaries

The segment analysis boundary is defined by the roadway right-of-way and the operational influence area of each boundary intersection. The influence area of a boundary intersection extends upstream from the intersection on each intersection leg. It includes all geometric features and traffic conditions that influence segment or intersection operation during the study period. For these reasons, the analysis boundaries should be established for each segment and intersection on the basis of the conditions present during the study period.

Travel Directions to Be Evaluated

Previous editions of the HCM have allowed the evaluation of one direction of travel along a segment (even when it served two-way traffic). That approach is retained in this edition for the analysis of bicycle and transit performance. For the analysis of pedestrian performance, this approach translates into the evaluation of sidewalk and street conditions on one side of the segment.

For the analysis of motorized vehicle performance, an analysis of only one travel direction (when the street serves two-way traffic) does not adequately recognize the interactions between vehicles at the boundary intersections and their influence on segment operation. For example, the motorized vehicle methodology in this edition of the HCM explicitly models the platoon formed by the signal at one end of the segment and its influence on the operation of the signal at the other end of the segment. For this reason, evaluation of both travel directions on a two-way segment is important.

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 duration of the analysis period is in the range of 0.25 to 1 h. The longer durations in this range are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1 h because traffic conditions are not typically steady for long time periods and because the adverse impact of short peaks in traffic demand may not be detected in the evaluation.

URBAN STREET SEGMENT DEFINED

Terminology

For the purpose of analysis, the roadway is separated into individual elements that are physically adjacent and operate as a single entity in serving travelers. Two elements are commonly found on an urban street system: points and links. A *point* is the boundary between links and is represented by an intersection or ramp terminal. A *link* is a length of roadway between two points. A link and its boundary points are referred to as a *segment.*

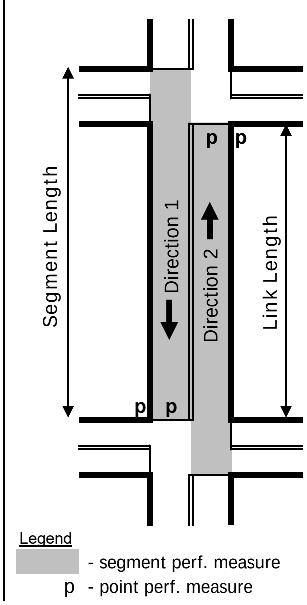
Points and Segments

A segment performance measure combines link performance and point performance.

The link and its boundary points must be evaluated together to provide an accurate indication of overall segment performance. For a given direction of travel along the segment, link and downstream point performance measures are combined to determine overall segment performance.

If the subject segment is within a coordinated signal system, the following rules apply when the segment boundaries are identified:

- A signalized intersection (or ramp terminal) is always used to define a segment boundary.
- Only intersections (or ramp terminals) at which the segment through movement is uncontrolled (e.g., a two-way STOP-controlled intersection) can exist along the segment between the boundaries.



If the subject segment is not within a coordinated signal system, the following rules apply when the segment boundaries are identified:

- An intersection (or ramp terminal) having a type of control that can impose on the segment through movement a legal requirement to stop or yield must always be used to define a segment boundary.
- An intersection (or ramp terminal) at which the segment through movement is uncontrolled (e.g., a two-way STOP-controlled intersection) may be used to define a segment boundary, but it is typically not done.

A midsegment traffic control signal provided for the exclusive use of pedestrians should not be used to define a segment boundary. This restriction reflects the fact that the methodologies described here were derived for and calibrated with data from street segments bounded by an intersection.

An access point intersection is an unsignalized intersection with one or two access point approaches to the segment. The approach can be a driveway or a public street. The through movements on the segment are uncontrolled at an access point intersection.

LOS CRITERIA

This subsection describes the LOS criteria for the motorized vehicle, pedestrian, bicycle, and transit modes. The criteria for the motorized vehicle mode are different from the criteria used for the other modes. Specifically, the criteria for the motorized vehicle mode are based on performance measures that are field-measurable and perceivable by travelers. With one exception, the criteria for the pedestrian and bicycle modes are based on scores reported by travelers indicating their perception of service quality. The exception is the pedestrian space measure (used with the pedestrian mode), which is field-measurable and perceivable by pedestrians. The criteria for the transit mode are based on measured changes in transit patronage due to changes in service quality.

Motorized Vehicle Mode

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

Two performance measures are used to characterize vehicular LOS for a given direction of travel along an urban street segment. One measure is travel speed for through vehicles. This speed reflects the factors that influence running time along the link and the delay incurred by through vehicles at the boundary intersection. The second measure is the volume-to-capacity ratio for the through movement at the downstream boundary intersection. These performance measures indicate the degree of mobility provided by the segment. The following paragraphs characterize each service level.

"Free-flow speed" is the average running speed of through vehicles traveling along a segment under low-volume conditions and not delayed by traffic control devices or other vehicles.

The "base free-flow speed" is defined to be the free-flow speed on longer segments.

LOS A describes primarily free-flow operation. Vehicles are completely unimpeded in their ability to maneuver within the traffic stream. Control delay at the boundary intersection is minimal. The travel speed exceeds 80% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS B describes reasonably unimpeded operation. The ability to maneuver within the traffic stream is only slightly restricted, and control delay at the boundary intersection is not significant. The travel speed is between 67% and 80% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS C describes stable operation. The ability to maneuver and change lanes at midsegment locations may be more restricted than at LOS B. Longer queues at the boundary intersection may contribute to lower travel speeds. The travel speed is between 50% and 67% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS D indicates a less stable condition in which small increases in flow may cause substantial increases in delay and decreases in travel speed. This operation may be due to adverse signal progression, high volume, or inappropriate signal timing at the boundary intersection. The travel speed is between 40% and 50% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS E is characterized by unstable operation and significant delay. Such operations may be due to some combination of adverse progression, high volume, and inappropriate signal timing at the boundary intersection. The travel speed is between 30% and 40% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS F is characterized by flow at extremely low speed. Congestion is likely occurring at the boundary intersection, as indicated by high delay and extensive queuing. The travel speed is 30% or less of the base free-flow speed, or the volume-to-capacity ratio is greater than 1.0.

Exhibit 18-1 lists the LOS thresholds established for the motorized vehicle mode on urban streets. The threshold value is interpolated when the base free-flow speed is between the values shown in the column headings of this exhibit. For example, the LOS A threshold for a segment with a base free-flow speed of 42 mi/h is 34 mi/h [= (42 – 40)/(45 – 40) × (36 – 32) + 32].

Exhibit 18-1
LOS Criteria: Motorized Vehicle Mode

LOS	Travel Speed Threshold by Base Free-Flow Speed (mi/h)							Volume-to-Capacity Ratio[a]
	55	50	45	40	35	30	25	
A	>44	>40	>36	>32	>28	>24	>20	≤ 1.0
B	>37	>34	>30	>27	>23	>20	>17	
C	>28	>25	>23	>20	>18	>15	>13	
D	>22	>20	>18	>16	>14	>12	>10	
E	>17	>15	>14	>12	>11	>9	>8	
F	≤17	≤15	≤14	≤12	≤11	≤9	≤8	
F	Any							> 1.0

Note: [a] Volume-to-capacity ratio of through movement at downstream boundary intersection.

Pedestrian, Bicycle, and Transit Modes

Historically, this manual has used a single performance measure as the basis for defining LOS. However, research documented in Chapter 5, Quality and Level-of-Service Concepts, indicates that travelers consider a wide variety of factors in assessing the quality of service provided to them. Some of these factors can be described as performance measures (e.g., speed), and others can be described as basic descriptors of the urban street character (e.g., sidewalk width). The methodologies for evaluating the pedestrian, bicycle, and transit modes combine these factors to determine the corresponding mode's LOS.

The Spatial Limits subsections of Sections 4 and 5 provide guidance on when to use segment- and link-based analyses for the pedestrian and bicycle modes, respectively.

Pedestrian quality of service can be evaluated for the segment, the link, or both. A segment-based pedestrian evaluation uses the worse of the LOS letters resulting from pedestrian space and the segment pedestrian LOS score to determine the overall segment pedestrian LOS. The left side of Exhibit 18-2 lists the threshold values associated with each LOS for the segment-based evaluation of the pedestrian travel mode. The LOS is determined by consideration of both the LOS score and the average pedestrian space on the sidewalk. The applicable LOS for an evaluation is determined from the table by finding the intersection of the row corresponding to the computed score value and the column corresponding to the computed space value.

Exhibit 18-2
LOS Criteria: Pedestrian Mode

Segment-Based Pedestrian LOS Score	Segment-Based LOS by Average Pedestrian Space (ft^2/p): >60	>40–60	>24–40	>15–24	>8.0–15[a]	≤8.0[a]	Link-Based Pedestrian LOS: Link-Based LOS Score	LOS
≤2.00	A	B	C	D	E	F	≤1.50	A
>2.00–2.75	B	B	C	D	E	F	>1.50–2.50	B
>2.75–3.50	C	C	C	D	E	F	>2.50–3.50	C
>3.50–4.25	D	D	D	D	E	F	>3.50–4.50	D
>4.25–5.00	E	E	E	E	E	F	>4.50–5.50	E
>5.00	F	F	F	F	F	F	>5.50	F

Note: [a] In cross-flow situations, the LOS E/F threshold is 13 ft^2/p. Chapter 4 describes the concept of "cross flow" and situations where it should be considered.

A link-based pedestrian evaluation uses the link pedestrian score to determine the overall link pedestrian LOS. The right side of Exhibit 18-2 lists the threshold values associated with each LOS for the link-based evaluation of the pedestrian travel mode. The LOS is determined by consideration of only the LOS score.

Exhibit 18-3 lists the range of scores that are associated with each LOS for the bicycle and transit modes. Similar to the pedestrian mode, bicycle LOS can be evaluated for the link, the segment, or both. Transit LOS is only evaluated for the segment.

Exhibit 18-3
LOS Criteria: Bicycle and Transit Modes

LOS	Segment-Based Bicycle LOS Score	Link-Based Bicycle LOS Score	Transit LOS Score
A	≤2.00	≤1.50	≤2.00
B	>2.00–2.75	>1.50–2.50	>2.00–2.75
C	>2.75–3.50	>2.50–3.50	>2.75–3.50
D	>3.50–4.25	>3.50–4.50	>3.50–4.25
E	>4.25–5.00	>4.50–5.50	>4.25–5.00
F	>5.00	>5.50	>5.00

The association between LOS score and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip along an urban street. 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 the respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

SCOPE OF THE METHODOLOGIES

This subsection identifies the conditions for which each methodology is applicable.

- *Boundary intersections.* All methodologies can be used to evaluate segment performance with signalized or two-way STOP-controlled boundary intersections. In the latter case, the cross street is STOP controlled. The motorized vehicle methodology can also be used to evaluate performance with all-way STOP- or YIELD-controlled (e.g., roundabout) boundary intersections.
- *Street types.* The four methodologies were developed with a focus on arterial and collector street conditions. If a methodology is used to

evaluate a local street, the performance estimates should be carefully reviewed for accuracy.

- *Flow conditions.* The four methodologies are based on the analysis of steady traffic conditions and are not well suited to the evaluation of unsteady conditions (e.g., congestion, cyclic spillback, signal preemption).
- *Target road users.* Collectively, the four methodologies were developed to estimate the LOS perceived by motorized vehicle drivers, pedestrians, bicyclists, and transit passengers. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers, automobile passengers, delivery truck drivers, or recreational vehicle drivers). However, it is likely that the perceptions of these other road users are reasonably well represented by the road users for whom the methodologies were developed.
- *Influences in the right-of-way.* A road user's perception of quality of service is influenced by many factors inside and outside of the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside of the right-of-way (e.g., buildings, parking lots, scenery, landscaped yards) that might influence a traveler's perspective. This approach was followed because factors outside of the right-of-way are not under the direct control of the agency operating the street.

3. MOTORIZED VEHICLE METHODOLOGY

This section describes the methodology for evaluating the capacity and quality of service provided to motorized vehicles on an urban street segment. Extensions to this methodology for evaluating more complex urban street operational elements are described in Chapter 30, Urban Street Segments: Supplemental.

SCOPE OF THE METHODOLOGY

The overall scope of the four methodologies was provided in Section 2. This section identifies the additional conditions for which the motorized vehicle methodology is applicable.

- *Target travel mode.* The motorized vehicle methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The methodology is not designed to evaluate the performance of other types of vehicles (e.g., golf carts, motorized bicycles).
- *Mobility focus.* The motorized vehicle methodology is intended to facilitate the evaluation of mobility. Accessibility to adjacent properties by way of motorized vehicle is not directly evaluated with this methodology. Regardless, a segment's accessibility should also be considered in evaluating its performance, especially if the segment is intended to provide such access. Oftentimes, factors that favor mobility reflect minimal levels of access and vice versa.

Spatial and Temporal Limits

Analysis Boundaries

An analysis of only one travel direction (when the street serves two-way traffic) does not adequately recognize the interactions between vehicles at the boundary intersections and their influence on segment operation. For this reason, evaluation of both travel directions on a two-way segment is important.

The analysis boundary for each boundary intersection is defined by the operational influence area of the intersection. It should include the most distant extent of any intersection-related queue expected to occur during the study period. For these reasons, the influence area for a signalized intersection is likely to extend at least 250 ft back from the stop line on each intersection leg.

Study Period and Analysis Period

The concepts of *study period* and *analysis period* are defined in Section 2 in general terms. They are defined more precisely in this subsection as they relate to the motorized vehicle methodology.

Exhibit 18-4 demonstrates three alternative approaches an analyst might use for a given evaluation. Other alternatives exist, and the study period can exceed 1 h. Approach A is the approach that has traditionally been used and, unless otherwise justified, is the approach that is recommended for use.

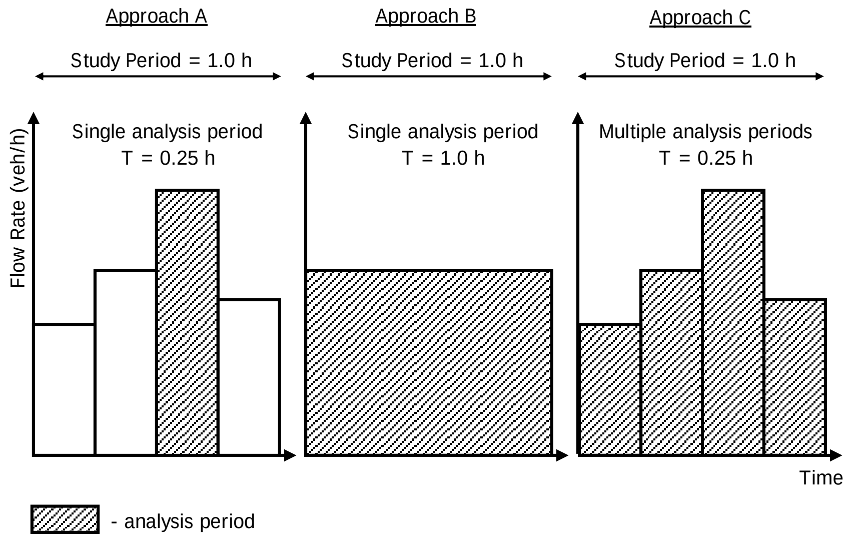


Exhibit 18-4
Three Alternative Study Approaches

The use of peak 15-min traffic multiplied by four is preferred for existing conditions when traffic counts are available. The use of a 1-h demand volume divided by a peak hour factor is preferred when volumes are projected or when hourly projected volumes have been added to existing volumes.

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*) a peak 15-min traffic count multiplied by four or (*b*) a 1-h demand volume divided by the peak hour factor. The former option is preferred for existing conditions when traffic counts are available; the latter option is preferred when hourly volumes are projected or when hourly projected volumes are added to existing volumes. Additional discussion on use of the peak hour factor is provided in the subsection titled Required Data and Sources.

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

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

Regardless of analysis period duration, a single-period analysis (i.e., Approach A or B) is typical for planning applications.

Multiple Time Period Analysis

If the analysis period’s demand volume exceeds capacity, a multiple time period analysis should be undertaken in which the study period includes an initial analysis period with no initial queue and a final analysis period with no residual queue. On a movement-by-movement and intersection-by-intersection basis, the initial queue for the second and subsequent periods is equal to the residual queue from the previous period. 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, the performance estimates for each period should be separately reported. 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 in reality some analysis periods have unacceptable operation.

Segment Length Considerations

The motorized vehicle methodology described in this section is not appropriate for the analysis of "short" segments that are bounded by signalized intersections. In contrast, the methodology described in Chapter 23, Ramp Terminals and Alternative Intersections, is appropriate for the analysis of short segments at signalized interchanges. The analyst may also consider using an alternative analysis tool that is able to model the operation of closely spaced intersections.

When a segment has a short length, the interaction between traffic movements and traffic control devices at the two boundary intersections is sufficiently complex that the motorized vehicle methodology may not provide an accurate indication of urban street performance. This complication can occur regardless of the type of control present at the two boundary intersections; however, the situation is particularly complicated when the two intersections are signalized.

Demand starvation occurs when a portion of the green at the downstream intersection is not used because the upstream intersection signalization prevents vehicles from reaching the stop line.

A short segment can experience "cyclic spillback." This spillback occurs when a queue extends back from one intersection into the other intersection (i.e., spills back) during a portion of each signal cycle and then subsides. A short segment can also experience "demand starvation." Demand starvation occurs when a portion of the green at the downstream intersection is not used because the upstream intersection signalization prevents vehicles from reaching the stop line. Demand starvation leads to the inefficient use of the downstream through phase and the retention of unserved vehicles on the approaches to the upstream intersection.

Specific conditions under which a segment bounded by signalized intersections should be considered "short" are difficult to define. As a general rule of thumb, cyclic spillback and demand starvation are unlikely to occur if the subject segment exceeds about 700 ft. They are also unlikely to occur on segments less than 700 ft *provided* that the following two conditions hold. First, the major traffic movement through the segment has coordinated signal timing that provides very favorable progression. Second, the coordinated traffic movement has about the same green-to-cycle-length ratio at each signal and each ratio is about 0.50 or larger. If the application of these rules to a specific segment indicates that cyclic spillback and starvation are unlikely to occur, the methodology described in this section can be used to evaluate the subject segment.

The methodology described in this section is applicable to segments having a length of 2 mi or less. This restriction is based on the fact that STOP-, YIELD-, or signal-controlled intersections are likely to have negligible effect on urban street operation when segment length exceeds 2 mi. Therefore, if a segment exceeds

2 mi in length, the analyst should evaluate it as an uninterrupted-flow highway segment with isolated intersections.

Performance Measures

Performance measures applicable to the motorized vehicle travel mode include travel speed, stop rate, and automobile traveler perception score. The latter measure provides an indication of the traveler's perception of service quality.

LOS is also considered a performance measure. It is useful for describing segment performance to elected officials, policy makers, administrators, or the public. LOS is based on travel speed and volume-to-capacity ratio.

Limitations of the Methodology

This subsection identifies the known limitations of the motorized vehicle methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific street segment, the analyst should consider using alternative methods or tools for the evaluation.

The motorized vehicle methodology does not account for the effect of the following conditions on urban street operation:

- Delay due to on-street parking maneuvers occurring along the link (see margin note for exceptions),
- Significant grade along the link,
- Queuing at the downstream boundary intersection backing up to and interfering with the operation of the upstream intersection or an access point intersection on a *cyclic* basis (e.g., as may occur at some interchange ramp terminals and closely spaced intersections),
- Stops incurred by segment through vehicles as a result of a vehicle ahead turning from the segment into an access point,
- Bicycles sharing a traffic lane with vehicular traffic, and
- Cross-street congestion or a railroad crossing that blocks through traffic.

The following parking-related effects are addressed in the methodology: (a) the effect on saturation flow rate of parking on the approach to a signalized intersection and (b) the effect on free-flow speed of parking stall presence along the street.

In addition, any limitations associated with the methodologies used to evaluate the intersections that bound the urban street segment are shared with this methodology. These limitations are listed in Chapters 19 to 23.

Lane Groups and Movement Groups

Lane group and *movement group* are phrases used to define combinations of intersection movements for the purpose of evaluating signalized intersection operation. These two terms are used extensively in the motorized vehicle methodology in Chapter 19, Signalized Intersections. They are also used in the motorized vehicle methodology when the boundary intersection is signalized.

The motorized vehicle methodology in Chapter 19 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 established to facilitate data entry to the methodology. In this regard, input data describing intersection traffic are traditionally specific to the movement (e.g., left-turn movement volume) and not specific to the lane (e.g., analysts rarely have the volume for a lane shared by left-turning and through vehicles). 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).

REQUIRED DATA AND SOURCES

Additional required input data, potential data sources, and default values for the roundabout segment methodology can be found in Section 9 of Chapter 30.

This subsection describes the input data needed for the motorized vehicle methodology. The required data are listed in Exhibit 18-5. They must be separately specified for each direction of travel on the segment and for each boundary intersection. The exhibit also lists default values that can be used if local data are not available (*3*).

The entries in the first column in Exhibit 18-5 indicate whether the input data are needed for a movement group at a boundary intersection, the overall intersection, or the segment. The input data needed to evaluate the boundary intersections are identified in the appropriate chapter (i.e., Chapters 19 to 23).

The data elements listed in Exhibit 18-5 do not include variables that are considered to represent calibration factors (e.g., acceleration rate). A calibration factor typically has a relatively narrow range of reasonable values or has 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.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 18-5. These data describe the motorized vehicle traffic stream traveling along the street during the analysis period.

Demand Flow Rate

The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period, divided by the analysis period duration. It is expressed as an hourly flow rate, but it may represent an analysis period shorter than 1 h. Guidance for estimating this rate is provided in the chapter that corresponds to the boundary intersection configuration (i.e., Chapters 19 to 23). The "count of vehicles" can be obtained from a variety of sources (e.g., from the field or as a forecast from a planning model).

Exhibit 18-5
Required Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
Traffic Characteristics Data		
Demand flow rate by movement group at boundary intersection (veh/h)	Field data, past counts	Must be provided
Access point flow rate by movement group (veh/h)	Field data, past counts	See discussion in text
Midsegment flow rate (veh/h)	Field data, past counts	Estimate by using demand flow rate at the downstream boundary int. approach
Geometric Data		
Number of lanes by movement group at boundary intersection	Field data, aerial photo	Must be provided
Upstream intersection width (ft)	Field data, aerial photo	Must be provided
Segment approach turn bay length at boundary intersection (ft)	Field data, aerial photo	Must be provided
Number of midsegment through lanes	Field data, aerial photo	Must be provided
Number of lanes at access points—segment approach	Field data, aerial photo	(*a*) Number of through lanes on approach = number of midsegment through lanes. (*b*) No right-turn lanes. (*c*) If median present, one left-turn lane per approach; otherwise, no left-turn lanes.
Number of lanes at access points—access point approach	Field data, aerial photo	One left-turn and one right-turn lane
Segment approach turn bay length at access points (ft)	Field data, aerial photo	40% of the access point spacing, where spacing = 2 × (5,280) / D_a in feet, but not more than 300 ft nor less than 50 ft
Segment length (ft)	Field data, aerial photo	Must be provided
Restrictive median length (ft)	Field data, aerial photo	Must be provided
Proportion of segment with curb (decimal)	Field data, aerial photo	1.0 (curb present on both sides of segment)
Number of access point approaches	Field data, aerial photo	See discussion in text
Proportion of segment with on-street parking (decimal)	Field data	Must be provided
Other Data		
Analysis period duration (h)	Set by analyst	0.25 h
Speed limit (mi/h)	Field data, road inventory	Must be provided
Performance Measure Data		
Through control delay at boundary intersection (s/veh)	HCM method output	Must be provided
Through stopped vehicles at boundary intersection (veh)	HCM method output	Must be provided
2nd- and 3rd-term back-of-queue size for through movement at boundary intersection (veh/lane)	HCM method output	Must be provided
Capacity by movement group at boundary intersection (veh/h)	HCM method output	Must be provided
Midsegment delay (s/veh)	Field data	0.0 s/veh
Midsegment stops (stops/veh)	Field data	0.0 stops/veh

Notes: Int. = intersection.
D_a = access point density on segment (points/mi).

Access Point Flow Rate

The access point flow rate is defined as the count of vehicles arriving at an access point intersection during the analysis period, divided by the analysis period duration. It is expressed as an hourly flow rate, but it may represent an analysis period shorter than 1 h. It should represent a demand flow rate.

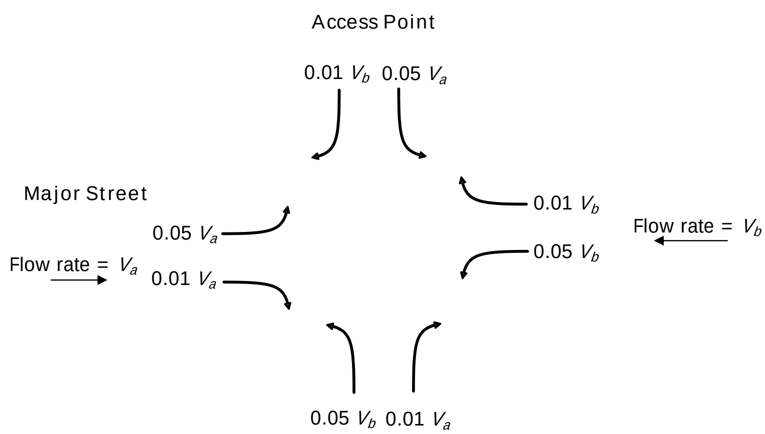
This flow rate is needed for all movements on each "active" access point approach and for all major-street movements at the intersection with one or more "active" access point approaches. An access point approach is considered to be *active* if its volume is sufficient to have some impact on segment operations during the analysis period. As a rule of thumb, an access point approach is considered active if it has an entering flow rate of 10 veh/h or more during the analysis period.

If the segment has many access point intersections that are considered inactive but collectively have some impact on traffic flow, those intersections can be combined into one equivalent active access point intersection. Each nonpriority movement at the equivalent access point intersection has a flow rate that is equal to the sum of the corresponding nonpriority movement flow rates of each of the individual inactive access points.

If a planning analysis is being conducted in which (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, each movement's hourly demand should 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, the default value in Exhibit 19-11 of Chapter 19 can be used.

Default value. The default access point flow rate can be estimated from the midsegment flow rate by using default turn proportions. These proportions are shown in Exhibit 18-6 for a typical access point intersection on an arterial street. The proportion of 0.05 for the left-turn movements can be reduced to 0.01 for a typical access point on a collector street. These proportions are appropriate for segments with a typical access point density. They are applicable to access points serving any public-oriented land use (this excludes single-family residential land use and undeveloped property).

Exhibit 18-6
Default Turn Proportions for Access Point Intersections



If one of the movements shown in Exhibit 18-6 does not exist at a particular access point intersection, its volume is not computed (its omission has no effect on the proportion used for the other movement flow rates). The flow rate for the through movement on an access point approach is not needed for the motorized vehicle methodology because this movement is considered to have negligible effect on major-street operation.

Midsegment Flow Rate

The midsegment flow rate is defined as the count of vehicles traveling along the segment during the analysis period, divided by the analysis period duration. It is expressed as an hourly flow rate, but it may represent an analysis period shorter than 1 h. This volume is specified separately for each direction of travel along the segment.

If one or more access point intersections exist along the segment, the midsegment flow rate should be measured at a location between these intersections (or between an access point and boundary intersection). The location chosen should be representative in terms of its having a flow rate similar to other locations along the segment. If the flow rate is believed to vary significantly along the segment, it should be measured at several locations and an average used in the methodology.

If a planning analysis is being conducted in which (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, each movement's hourly demand should be divided by the peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor used should be based on local traffic peaking trends.

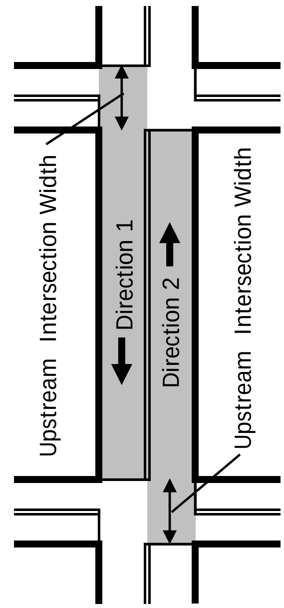
Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 18-5. These data describe the geometric elements of the segment or intersections that are addressed in the motorized vehicle methodology.

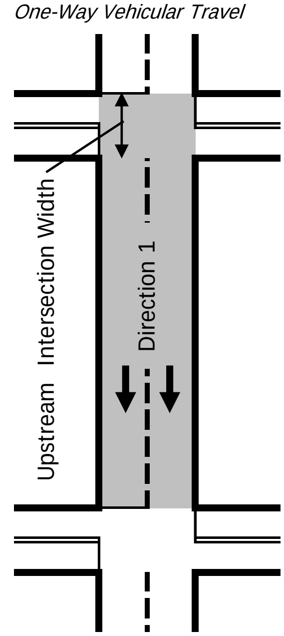
Number of Lanes at Boundary Intersection

The number of lanes at the boundary intersection is the count of lanes that are 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, the turn movement is indicated to have 0 lanes.

Two-Way Vehicular Travel



One-Way Vehicular Travel



Upstream Intersection Width

The intersection width applies to the upstream boundary intersection for a given direction of travel and is the effective width of the cross street. On a two-way street, it is the distance between the stop (or yield) line for the two opposing segment through movements at the boundary intersection, as measured along the centerline of the segment. On a one-way street, it is the distance from the stop line to the far side of the most distant traffic lane on the cross street.

Turn Bay Length at Boundary Intersection

Turn bay length is the length of the bay at the boundary intersection 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 differing lengths, 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, 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 their associated left-turning vehicles that store in the two-way left-turn lane.

Number of Through Lanes on the Segment

The number of through lanes on the segment is the count of lanes that extend for the length of the segment and serve through vehicles (even if a lane is dropped or added at a boundary intersection). This count is specified separately for each direction of travel along the segment. A lane provided for the exclusive use of turning vehicles is not included in this count.

If there is a midsegment lane restriction, the number of through lanes equals the number of lanes through the restriction. For example, if a work zone is present and it requires one through lane to be closed, the number of through lanes equals the count of through lanes that remain open through the work zone (and does not include the count of lanes that are closed).

Number of Lanes at Access Points

The number of lanes at an access point intersection is the count of lanes that are provided for each traffic movement at the intersection. The method for determining this number follows the same guidance provided in a previous paragraph for the number of lanes at boundary intersections.

This input data element is needed for all movements on each active access point approach and for all major-street movements at the intersection with one or more active access point approaches. Guidance for determining whether an access point is "active" is provided in the section titled Access Point Flow Rate.

Turn Bay Length at Access Points

Turn bay length is the length of the bay at the access point intersection for which the lanes have full width and in which queued vehicles can be stored. This length is needed for both segment approaches to the access point intersection. The method for determining this length follows the same guidance provided in a previous paragraph for turn bay length at boundary intersections.

This input data element is needed for all major-street turn movements at the intersection with one or more active access point approaches. Guidance for determining whether an access point is "active" is provided in the section titled Access Point Flow Rate.

Segment Length

Segment length is the distance between the boundary intersections that define the segment. The point of measurement at each intersection is the stop line, the yield line, or the functional equivalent in the subject direction of travel.

This length is measured along the centerline of the street. If it differs in the two travel directions, an average length is used.

The *link length* is used in some calculations. It is computed as the segment length minus the width of the upstream boundary intersection.

Restrictive Median Length

The restrictive median length is the length of street with a restrictive median (e.g., raised curb). This length is measured from median nose to median nose along the centerline of the street. It does not include the length of any median openings on the street.

Proportion of Segment with Curb

The proportion of the segment with curb is the proportion of the link length with curb along the right side of the segment that is within 4 ft of the traveled way (i.e., within 4 ft of the nearest edge of traffic lane). This proportion is computed as the length of street with a curb present (and within 4 ft) divided by the link length. The length of street with a curb present is measured from the start of the curbed cross section to the end of the curbed cross section on the link. The width of driveway openings is *not* deducted from this length. This proportion is computed separately for each direction of travel along the segment.

Number of Access Point Approaches

The number of access point approaches along a segment is the count of *all* unsignalized driveway and public-street approaches to the segment, regardless of whether the access point is considered to be active. This number is counted separately for each side of the segment. It must equal or exceed the number of active access point approaches for which delay to segment through vehicles is computed. Guidance for determining whether an access point is “active” is provided in the section titled Access Point Flow Rate. If the downstream boundary intersection is unsignalized, its cross-street approach on the right-hand side (in the direction of travel) is included in the count.

Default value. When the number of access points is not known, it can be estimated from a specified access point density by using the following equation:

Equation 18-1

$$N_{ap,s} = 0.5\,\frac{D_a\,L}{5{,}280}$$

where $N_{ap,s}$ is the number of access point approaches on the right side in the subject direction of travel (points), D_a is the access point density on the segment (points/mi), and L is the segment length (feet). A default number of access points can be determined from the default access point density obtained from Exhibit 18-7.

Exhibit 18-7
Default Access Point Density Values

Area Type	Median Type	Default Access Point Density (points/mi) by Speed Limit (mi/h)						
		25	30	35	40	45	50	55
Urban	Restrictive	62	50	41	35	30	26	22
	Other	73	61	52	46	41	37	33
Suburban or rural	Restrictive	40	27	19	12	7	3	0
	Other	51	38	30	23	18	14	11

Proportion of Segment with On-Street Parking

The proportion of the segment with on-street parking is the proportion of the link length with parking stalls (either marked or unmarked) available along the right side of the segment. This proportion is computed as the length of street with parking stalls divided by the link length. Parking stalls considered include those described as having either a parallel or an angle design. This proportion is separately computed for each direction of travel along the segment.

Other Data and Performance Measures

This subsection describes the data listed in Exhibit 18-5 that are categorized as "other data" or "performance measure data."

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. Its duration is in the range of 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should use caution in interpreting the results from an analysis period of 1 h or more because the adverse impact of short peaks in traffic demand may not be detected.

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

Operational analysis. A 15-min analysis period should be used for operational analyses. This duration will accurately capture the adverse effects of demand peaks.

Planning analysis. A 15-min analysis period is used for most planning analyses. However, a 1-h analysis period can be used, if appropriate.

Speed Limit

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 segment and (*b*) consistent with agency policy concerning specification of speed limits. If the posted speed limit is known not to satisfy these assumptions, the speed limit value that is input to the methodology should be adjusted so that it is consistent with the assumptions.

Through Control Delay

The through control delay is the control delay to the through movement at the downstream boundary intersection. It is computed by using the appropriate procedure provided in one of Chapters 19 to 23, depending on the type of control used at the intersection.

If the intersection procedure provides delay by lane groups and the through movement is served in two or more lane groups, the through-movement delay is computed as the weighted sum of the individual lane-group delays, where the weight for a lane group is its proportion of through vehicles.

Through Stopped Vehicles and Second- and Third-Term Back-of-Queue Size

Three variables are needed for the calculation of stop rate when the downstream boundary intersection is signalized. They apply to the through lane group at this intersection. A procedure for computing the number of fully stopped vehicles N_f, second-term back-of-queue size Q_2, and third-term back-of-queue size Q_3 is provided in Chapter 31, Signalized Intersections: Supplemental.

If the procedure provides the stop rate by lane groups and the through movement is served in two or more lane groups, the through-movement stop rate is computed as the weighted sum of the individual lane-group stop rates, where the weight for a lane group is its proportion of through vehicles.

Capacity

The capacity of a movement group is the maximum number of vehicles that can discharge from a queue during the analysis period, divided by the analysis period duration. This value is needed for the movements entering the segment at the upstream boundary intersection and for the movements exiting the segment at the downstream boundary intersection. With one exception, it is computed by using the appropriate procedure provided in one of Chapters 19 to 23, depending on the type of control used at the intersection. Chapter 20, Two-Way STOP-Controlled Intersections, does not provide a procedure for estimating the capacity of the uncontrolled through movement, but this capacity can be estimated by using Equation 18-2.

$$c_{th} = 1{,}800\left(N_{th} - 1 + p^*_{0,j}\right)$$

Equation 18-2

where

c_{th} = through-movement capacity (veh/h),

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

$p^*_{0,j}$ = probability that there will be no queue in the inside through lane.

The probability $p^*_{0,j}$ is computed by using Equation 20-43 in Chapter 20. It is equal to 1.0 if a left-turn bay is provided for left turns from the major street.

If the procedure in Chapters 19 to 23 provides capacity by lane group and the through movement is served in two or more lane groups, the through-movement capacity is computed as the weighted sum of the individual lane-group capacities, where the weight for a lane group is its proportion of through vehicles. A similar approach is used to compute the capacity for a turn movement.

Midsegment Delay and Stops

Through vehicles traveling along a segment can encounter a variety of situations that cause them to slow slightly or even come to a stop. These encounters delay the through vehicles and cause their segment running time to increase. Situations that can cause this delay include

- Vehicles turning from the segment into an access point approach,
- Pedestrians crossing at a midsegment crosswalk,

- Vehicles maneuvering into or out of an on-street parking space,
- Double-parked vehicles blocking a lane, and
- Vehicles in a dropped lane that are merging into the adjacent lane.

A procedure is provided in the methodology for estimating the delay due to vehicles turning left or right into an access point approach. This edition of the HCM does not include procedures for estimating the delay or stops due to the other sources listed. If they exist on the subject segment, they must be estimated by the analyst and input to the methodology.

OVERVIEW OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of the urban street segment in terms of its service to motorized vehicles. The methodology is computationally intense and requires software to implement. The intensity stems from the need to model the traffic movements that enter or exit the segment in terms of their interaction with each other and with the traffic control elements of the boundary intersection.

A planning-level analysis application for evaluating segment performance is provided in Section 5 of Chapter 30, Urban Street Segments: Supplemental. This method is not computationally intense and can be applied by using hand calculations.

Each travel direction along the segment is separately evaluated. *Unless otherwise stated, all variables are specific to the subject direction of travel.*

The methodology has been developed to evaluate motorized vehicle performance for a street segment bounded by intersections that can have a variety of control types. The focus of the discussion in this subsection is on the use of the methodology to evaluate a coordinated signal system because this type of control is the most complex. However, as appropriate, the discussion is extended to describe how key elements of this methodology can be used to evaluate motorized vehicle performance in noncoordinated systems.

The objective of this overview is to introduce the analyst to the calculation process and to discuss the key analytic procedures. This objective is achieved by outlining the procedures that make up the methodology while highlighting important equations, concepts, and interpretations. A more detailed discussion of these procedures is provided in Sections 2, 3, and 4 of Chapter 30, Urban Street Segments: Supplemental.

The computational engine developed by the Transportation Research Board Committee on Highway Capacity and Quality of Service represents the most detailed description of this methodology. Additional information about this engine is provided in Section 7 of Chapter 30.

A methodology for evaluating the performance of the motorized vehicle mode on an urban street segment bounded by one or more roundabouts is provided in Section 9 of Chapter 30.

Exhibit 18-8 illustrates the calculation framework of the motorized vehicle methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process flows from top to bottom in the exhibit. These calculations are described more fully in the next section.

The framework illustrates the calculation process as applied to two system types: coordinated and noncoordinated. The analysis of coordinated systems recognizes the influence of an upstream signalized intersection on the performance of the street segment. The analysis of noncoordinated systems is based on the assumption that arrivals to a boundary intersection are random.

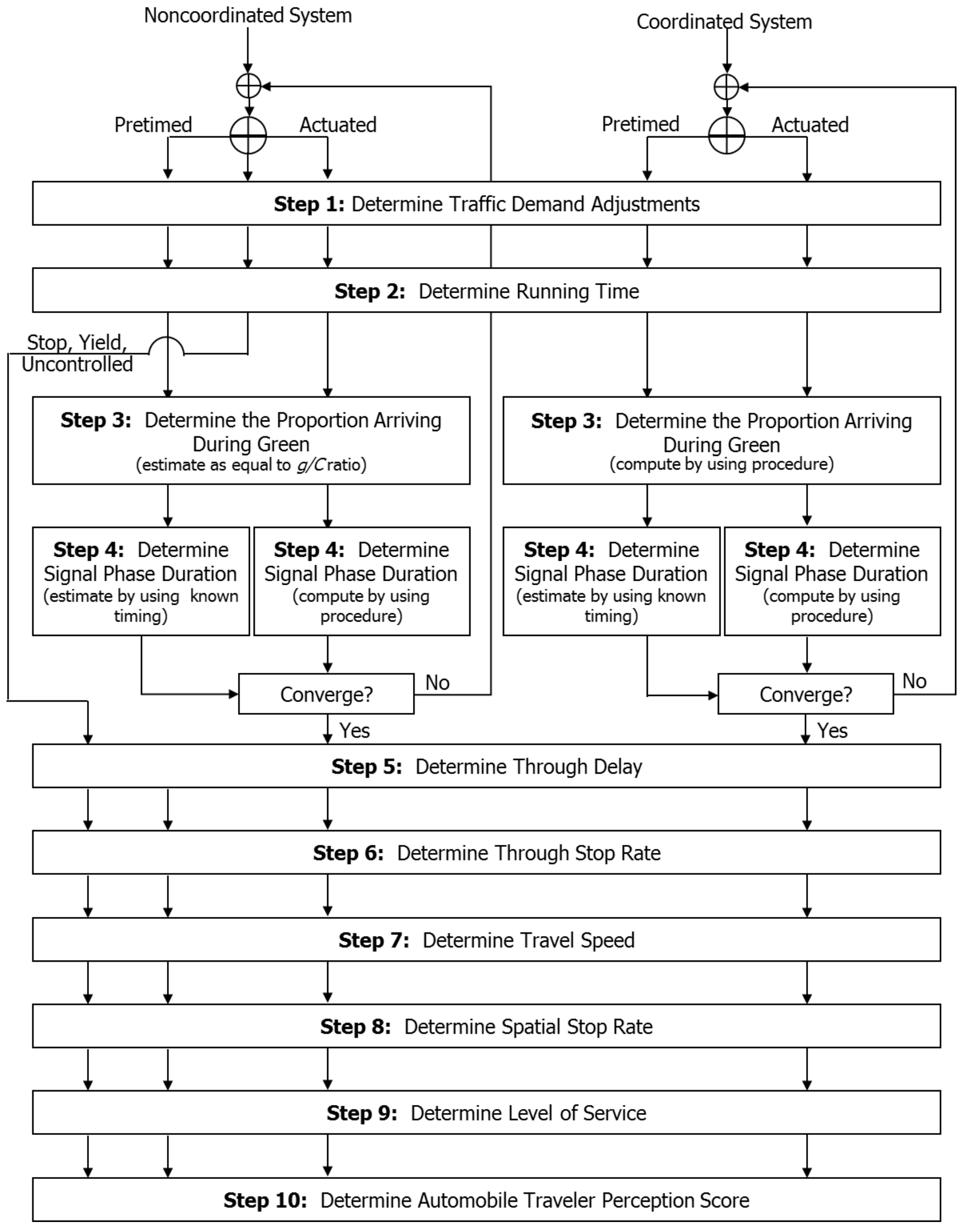
Details on the methodology for segments with roundabouts as boundary intersections can be found in Section 9 of Chapter 30.

The framework is further subdivided into the type of traffic control used at the intersections that bound the segment. This approach recognizes that a boundary intersection can be signalized, two-way STOP-controlled, all-way STOP-controlled, or a roundabout. Although not indicated in the exhibit, the boundary intersection could also be an interchange ramp terminal.

The methodology is shown to be iterative within Steps 1 to 4, with convergence achieved when the predicted discharge volume, phase duration, and capacity from successive iterations are effectively in agreement. Several iterations are typically needed for coordinated systems. In contrast, only one iteration is needed for noncoordinated systems unless there is a downstream lane closure (e.g., a midsegment work zone), in which case multiple iterations are needed to ensure that the vehicles discharged upstream of the lane closure do not exceed the lane closure capacity. The procedure for analyzing midsegment lane restrictions is described in Section 3 of Chapter 30.

Procedures in other chapters are needed to evaluate an urban street segment. For example, the procedure in Section 3 of Chapter 19 for computing actuated phase duration is needed for the analysis of actuated intersections on both coordinated and noncoordinated segments. Also, the procedure in Section 3 of Chapter 19 for computing control delay is needed for the estimation of segment through-movement delay. The capacity and control delay estimation procedures for roundabouts and all-way STOP-controlled intersections are needed from their respective chapters for the analysis of noncoordinated segments.

Exhibit 18-8
Motorized Vehicle Methodology for Urban Street Segments



COMPUTATIONAL STEPS

Step 1: Determine Traffic Demand Adjustments

During this step, various adjustments are undertaken to ensure that the volumes evaluated accurately reflect segment traffic conditions. The adjustments include (*a*) limiting entry to the segment because of capacity constraint, (*b*) balancing the volumes entering and exiting the segment, and (*c*) mapping entry-to-exit flow paths by using an origin–destination matrix. Also during this step, a check is made for the occurrence of spillback from a turn bay or from one segment into another segment.

The procedures for making the aforementioned adjustments and checks are described in Section 2 of Chapter 30. These adjustments and checks are not typically used for planning and preliminary engineering analyses. If spillback occurs, the sustained spillback procedure should be used. It is described in Section 3 of Chapter 29, Urban Street Facilities: Supplemental.

Capacity Constraint

When the demand volume for an intersection traffic movement exceeds its capacity, the discharge volume from the intersection is restricted (or metered). When this metering occurs for a movement that enters the subject segment, the volume arriving at the downstream signal is reduced below the unrestricted value.

To determine whether metering occurs, the capacity of each upstream movement that discharges into the subject segment must be computed and then checked against the associated demand volume. If this volume exceeds movement capacity, the volume entering the segment must be reduced to equal the movement capacity.

Volume Balance

Volume balance describes a condition in which the combined volume from all movements entering a segment equals the combined volume exiting the segment, in a given direction of travel. The segment is balanced when entering volume equals exit volume for both directions of travel. Unbalanced volumes often exist in turn movement counts when the count at one intersection and that at the adjacent intersection are taken at different times. They are also likely when access point intersections exist but their volume is not counted.

The accuracy of the performance evaluation may be adversely affected if the volumes are not balanced. The extent of the impact is based on the degree to which the volumes are unequal. To balance the volumes, the methodology assumes that the volume for each movement entering the segment is correct and adjusts the volume for each movement exiting the segment in a proportional manner so that a balance is achieved. The exiting volumes computed in this manner represent a best estimate of the actual *demand* volumes, such that the adjustment process does not preclude the possibility of queue buildup by one or more exit movements at the downstream boundary intersection during the analysis period.

Origin–Destination Distribution

The volume of traffic that arrives at a downstream intersection for a given downstream movement represents the combined volume from each upstream point of entry weighted by its percentage contribution to the downstream movement. The distribution of these contribution percentages between each upstream and downstream pair is represented as an origin–destination distribution matrix.

The concept of an origin–destination distribution matrix is illustrated by example. Consider the segment shown in Exhibit 18-9, which has four entry points and four exit points. There are three entry volumes at upstream Intersection A that contribute to three exit volumes at downstream Intersection B. There are also an entrance and an exit volume at the access point intersection located between the two intersections. The volume entering the segment, 1,350 veh/h, is the same as that exiting the segment; thus, there is volume balance for this example segment. The origin–destination distribution matrix for this sample street segment is shown in Exhibit 18-10.

Exhibit 18-9
Entry and Exit Volume on Example Segment

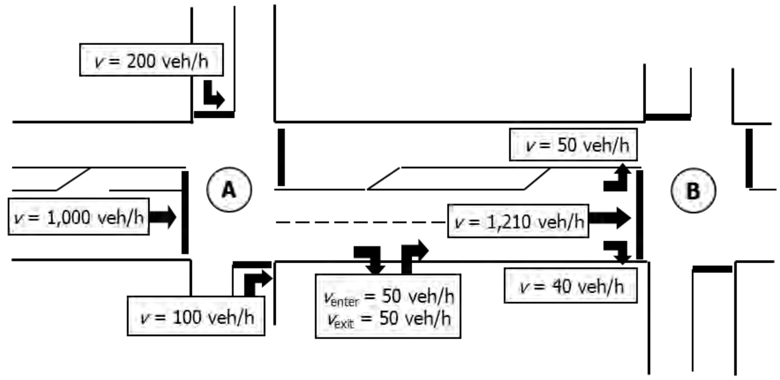


Exhibit 18-10
Example Origin–Destination Distribution Matrix

Origin Volume by Movement (veh/h)				Destination Volume	
Left	Through	Right	Access Point	Movement	Total Volume (veh/h)
2	46	2	0	Left	50
188	877	95	50	Through	1,210
3	36	1	0	Right	40
7	41	2	0	Access point	50
200	1,000	100	50		1,350

The column totals in the last row of Exhibit 18-10 correspond to the entry volumes shown in Exhibit 18-9. The row totals in the last column of Exhibit 18-10 indicate the exit volumes. The individual cell values indicate the volume contribution of each upstream movement to each downstream movement. For example, of the 1,000 through vehicles that enter the segment, 877 depart the segment as a through movement, 46 depart as a left-turn movement, and so on. The volumes in the individual cells are sometimes expressed as a proportion of the column total.

The motorized vehicle methodology computes one origin–destination matrix for movements between the upstream boundary intersection and a downstream junction (i.e., either an access point or the downstream boundary intersection). When the boundary intersections are signalized, the matrix for movements between the upstream and downstream boundary intersections is used to compute the proportion of vehicles arriving during the green indication for each exit movement. The matrix for movements between the upstream boundary intersection and a downstream access point is used to compute the proportion of time that a platoon is passing through the access point and effectively blocking nonpriority movements from entering or crossing the street.

Spillback Occurrence

Segment spillback can be characterized as one of two types: cyclic and sustained. Cyclic spillback occurs when the downstream boundary intersection is signalized and its queue backs into the upstream intersection as a result of queue growth during the red indication. When the green indication is presented, the queue dissipates and spillback is no longer present for the remainder of the cycle. This type of spillback can occur on short street segments with relatively long signal cycle lengths. The methodology may not provide a reliable estimate of segment performance if cyclic spillback occurs.

Sustained spillback occurs at some point during the analysis period and is a result of oversaturation (i.e., more vehicles discharging from the upstream intersection than can be served at the subject downstream intersection). The queue does not dissipate at the end of each cycle. Rather, it remains present until the downstream capacity is increased or the upstream demand is reduced.

The preceding discussion has focused on segment spillback; however, the concepts are equally applicable to turn bay spillback. In this case, the queue of turning vehicles exceeds the bay storage and spills back into the adjacent lane that is used by other vehicular movements.

The occurrence of both sustained segment and bay spillback must be checked during this step. A procedure is described in Section 3 of Chapter 30 for this purpose. If the spillback does not occur during the analysis period (i.e., it never occurs, or it occurs after the analysis period), the methodology will provide a reliable estimate of segment performance.

A procedure is described in Section 3 of Chapter 29 for evaluating the occurrence of sustained segment spillback during the analysis period.

If turn bay spillback occurs during the analysis period, the methodology may not yield reliable performance estimates. In this situation, the analyst should consider either (*a*) reducing the analysis period so that it ends before spillback occurs or (*b*) using an alternative analysis tool that can model the effect of spillback conditions.

Step 2: Determine Running Time

A procedure for determining segment running time is described in this step. This procedure includes the calculation of free-flow speed, a vehicle proximity adjustment factor, and the additional running time due to midsegment delay sources. Each calculation is discussed in the following subparts, which culminate with the calculation of segment running time.

A. Determine Free-Flow Speed

Free-flow speed is the average running speed of through vehicles traveling along a segment under low-volume conditions and not delayed by traffic control devices or other vehicles. It reflects the effect of the street environment on driver speed choice. Elements of the street environment that influence this choice under free-flow conditions include speed limit, access point density, median type, curb presence, and segment length.

The determination of free-flow speed is based on the calculation of base free-flow speed and an adjustment factor for signal spacing. These calculations are described in the next few paragraphs, which culminate in the calculation of free-flow speed.

Base Free-Flow Speed

The base free-flow speed is defined to be the free-flow speed on longer segments. It includes the influence of speed limit, access point density, median type, curb presence, and on-street parking presence. It is computed by using Equation 18-3. Alternatively, it can be measured in the field by using the technique described in Section 6 of Chapter 30.

Equation 18-3

$$S_{fo} = S_{calib} + S_0 + f_{cs} + f_A + f_{pk}$$

where

S_{fo} = base free-flow speed (mi/h),

S_{calib} = base free-flow speed calibration factor (mi/h),

S_0 = speed constant (mi/h),

f_{CS} = adjustment for cross section (mi/h),

f_A = adjustment for access points (mi/h), and

f_{pk} = adjustment for on-street parking (mi/h).

The speed constant and adjustment factors used in Equation 18-3 are listed in Exhibit 18-11 (*1*). Equations provided in the table footnote can also be used to compute these adjustment factors for conditions not shown in the exhibit.

Exhibit 18-11
Base Free-Flow Speed Adjustment Factors

Speed Limit (mi/h)	Speed Constant S_0 (mi/h)[a]	Median Type	Percent with Restrictive Median (%)	Adjustment for Cross Section f_{CS} (mi/h)[b]	
				No Curb	Curb
25	37.4	Restrictive	20	0.3	−0.9
30	39.7		40	0.6	−1.4
35	42.1		60	0.9	−1.8
40	44.4		80	1.2	−2.2
45	46.8		100	1.5	−2.7
50	49.1	Nonrestrictive	Not applicable	0.0	−0.5
55	51.5	No median	Not applicable	0.0	−0.5

Access Density D_a (points/mi)	Adjustment for Access Points f_A by Lanes N_{th} (mi/h)[c]			Percent with On-Street Parking (%)	Adjustment for Parking (mi/h)[d]
	1 Lane	2 Lanes	3 Lanes		
0	0.0	0.0	0.0	0	0.0
2	−0.2	−0.1	−0.1	20	−0.6
4	−0.3	−0.2	−0.1	40	−1.2
10	−0.8	−0.4	−0.3	60	−1.8
20	−1.6	−0.8	−0.5	80	−2.4
40	−3.1	−1.6	−1.0	100	−3.0
60	−4.7	−2.3	−1.6		

Notes: [a] S_0 = 25.6 + 0.47S_{pl}, where S_{pl} = posted speed limit (mi/h).
[b] f_{CS} = 1.5 p_{rm} – 0.47 p_{curb} – 3.7 p_{curb} p_{rm}, where p_{rm} = proportion of link length with restrictive median (decimal) and p_{curb} = proportion of segment with curb on the right-hand side (decimal).
[c] f_A = −0.078 D_a / N_{th} with D_a = 5,280 ($N_{ap,s}$ + $N_{ap,o}$)/(L – W_i), where D_a = access point density on segment (points/mi); N_{th} = number of through lanes on the segment in the subject direction of travel (ln); $N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points); $N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points); L = segment length (ft); and W_i = width of signalized intersection (ft).
[d] f_{pk} = −3.0 × proportion of link length with on-street parking available on the right-hand side (decimal).

Equation 18-3 has been calibrated by using data for many urban street segments collectively located throughout the United States, so the default value of 0.0 mi/h for S_{calib} is believed to yield results that are reasonably representative of driver behavior in most urban areas. However, if desired, a locally representative value can be determined from field-measured estimates of the base free-flow speed for several street segments. The local default value can be established for typical street segments or for specific street types. This calibration factor is determined as the one value that provides a statistically based best-fit between the prediction from Equation 18-3 and the field-measured estimates. A procedure for estimating the base free-flow speed from field data is described in Section 6 of Chapter 30.

Adjustment for Signal Spacing

Empirical evidence suggests that a shorter segment length (when defined by signalized boundary intersections) tends to influence the driver's choice of free-flow speed (*1*). Shorter segments have been found to have a slower free-flow speed, all other factors being the same. Equation 18-4 is used to compute the value of an adjustment factor that accounts for this influence.

$$f_L = 1.02 - 4.7 \frac{S_{fo} - 19.5}{\max(L_s, 400)} \leq 1.0$$

Equation 18-4

where

f_L = signal spacing adjustment factor,

S_{fo} = base free-flow speed (mi/h), and

L_s = distance between adjacent signalized intersections (ft).

Equation 18-4 was derived by using signalized boundary intersections. For more general applications, the definition of distance L_s is broadened so that it equals the distance between the two intersections that (*a*) bracket the subject segment and (*b*) have a type of control that can impose a legal requirement to stop or yield on the subject through movement.

Free-Flow Speed

The predicted free-flow speed is computed by using Equation 18-5 on the basis of estimates of base free-flow speed and the signal spacing adjustment factor. Alternatively, it can be entered directly by the analyst. It can also be measured in the field by using the technique described in Chapter 30.

$$S_f = S_{fo}\, f_L \geq S_{pl}$$

Equation 18-5

where S_f is the free-flow speed (mi/h), S_{pl} is the posted speed limit, and all other variables are as previously defined. The speed obtained from Equation 18-5 is always greater than or equal to the speed limit.

B. Compute Adjustment for Vehicle Proximity

The proximity adjustment factor adjusts the free-flow running time to account for the effect of traffic density. The adjustment results in an increase in running time (and corresponding reduction in speed) with an increase in volume. The reduction in speed is a result of shorter headways associated with the higher volume and drivers' propensity to be more cautious when headways are short. Equation 18-6 is used to compute the proximity adjustment factor.

Equation 18-6

$$f_v = \frac{2}{1 + \left(1 - \frac{v_m}{52.8\, N_{th}\, S_f}\right)^{0.21}}$$

where

f_v = proximity adjustment factor,

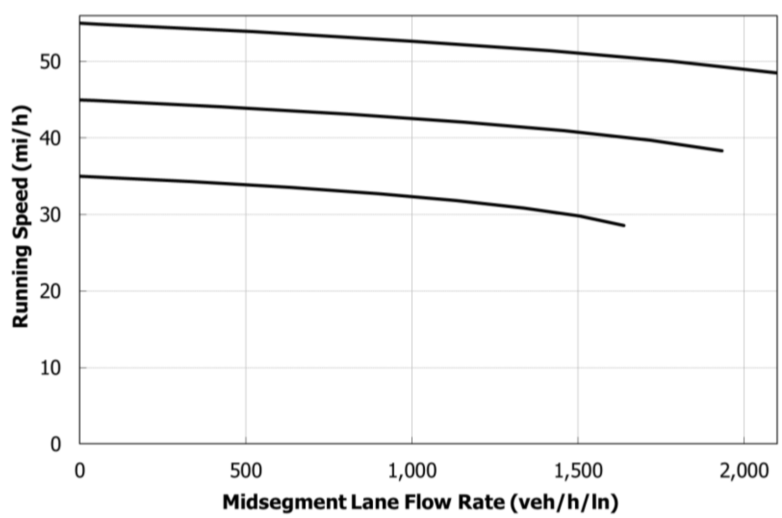
v_m = midsegment demand flow rate (veh/h),

N_{th} = number of through lanes on the segment in the subject direction of travel (ln), and

S_f = free-flow speed (mi/h).

The relationship between running speed [= (3,600 L)/(5,280 t_R), where L is the segment length in feet and t_R is the segment running time in seconds] and volume for an urban street segment is shown in Exhibit 18-12. Trend lines are shown for three specific free-flow speeds. At a flow rate of 1,000 vehicles per hour per lane (veh/h/ln), each trend line shows a reduction of about 2.5 mi/h relative to the free-flow speed. The trend lines extend beyond 1,000 veh/h/ln. However, a volume in excess of this amount is unlikely to be experienced on a segment bounded by intersections at which the through movement is regulated by a traffic control device.

Exhibit 18-12
Speed–Flow Relationship for Urban Street Segments



C. Compute Delay due to Turning Vehicles

Vehicles turning from the subject street segment into an access point approach can cause a delay to following through vehicles. For right-turn vehicles, the delay results when the following vehicles' speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point intersection. Delay due to left-turning vehicles occurs primarily on undivided streets; however, it can occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane. A procedure for computing this delay at each access point intersection is described in Section 4 of Chapter 30.

For planning and preliminary engineering analyses, Exhibit 18-13 can be used to estimate the delay due to turning vehicles at one representative access point intersection by using a midsegment volume that is typical for all such access points. The values in the exhibit represent the delay to through vehicles due to left and right turns at one access point intersection. The selected value is multiplied by the number of access point intersections on the segment to estimate delay due to left and right turns (= $\sum d_{ap}$ in Equation 18-7).

Exhibit 18-13
Delay due to Turning Vehicles

Midsegment Volume (veh/h/ln)	Through Vehicle Delay (s/veh/pt) by Number of Through Lanes		
	1 Lane	2 Lanes	3 Lanes
200	0.04	0.04	0.05
300	0.08	0.08	0.09
400	0.12	0.15	0.15
500	0.18	0.25	0.15
600	0.27	0.41	0.15
700	0.39	0.72	0.15

The values listed in Exhibit 18-13 represent 10% left turns and 10% right turns from the segment at the access point intersection. If the actual turn percentages are less than 10%, the delays can be reduced proportionally. For example, if the subject access point has 5% left turns and 5% right turns, the values listed in the exhibit should be multiplied by 0.5 (= 5/10). Also, if a turn bay of adequate length is provided for one turn movement but not the other, the values listed in the exhibit should be multiplied by 0.5. If both turn movements are provided a bay of adequate length, the delay due to turns can be assumed to equal 0.0 second per vehicle per access point (s/veh/pt).

D. Estimate Delay due to Other Sources

Numerous other factors could cause a driver traveling along a segment to reduce speed or to incur delay. For example, a vehicle that is completing a parallel parking maneuver may cause following vehicles to incur some delay. Also, vehicles that yield to pedestrians at a midsegment crosswalk may incur delay. Finally, bicyclists riding in a traffic lane or an adjacent bicycle lane may directly or indirectly cause vehicular traffic to adopt a lower speed.

Among the many sources of midsegment delay, the motorized vehicle methodology only includes procedures for estimating the delay due to turning vehicles. However, if the delay due to other sources is known or estimated by other means, it can be included in the equation to compute running time.

E. Compute Segment Running Time

Equation 18-7 is used to compute segment running time on the basis of consideration of through movement control at the boundary intersection, free-flow speed, vehicle proximity, and various midsegment delay sources.

$$t_R = \frac{6.0 - l_1}{0.0025\ L} f_x + \frac{3{,}600\ L}{5{,}280\ S_f} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{\text{other}}$$

Equation 18-7

with

Equation 18-8

$$f_x = \begin{cases} 1.00 & \text{(signalized or STOP-controlled through movement)} \\ 0.00 & \text{(uncontrolled through movement)} \\ \min\left[\frac{v_{th}}{c_{th}}, 1.00\right] & \text{(YIELD-controlled through movement)} \end{cases}$$

where

t_R = segment running time (s);

l_1 = start-up lost time = 2.0 if signalized, 2.5 if STOP or YIELD controlled (s);

L = segment length (ft);

f_x = control-type adjustment factor;

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

c_{th} = through-movement capacity (veh/h);

$d_{ap,i}$ = delay due to left and right turns from the street into access point intersection i (s/veh);

N_{ap} = number of influential access point approaches along the segment = $N_{ap,s}$ + $p_{ap,lt} N_{ap,o}$ (points);

$N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points);

$N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points);

$p_{ap,lt}$ = proportion of $N_{ap,o}$ that can be accessed by a left turn from the subject direction of travel; and

d_{other} = delay due to other sources along the segment (e.g., curb parking or pedestrians) (s/veh).

The variables l_1, f_x, v_{th}, and c_{th} used with the first term in Equation 18-7 apply to the through movement exiting the segment at the boundary intersection. This term accounts for the time required to accelerate to the running speed, less the start-up lost time. The divisor in this term is an empirical adjustment that minimizes the contribution of this term for longer segments. It partially reflects a tendency for drivers to offset this added time by adopting slightly higher midsegment speeds than reflected in the start-up lost time estimate.

Step 3: Determine the Proportion Arriving During Green

This step applies to the downstream boundary intersection when the operation of a signalized urban street segment is evaluated. If the downstream boundary intersection is not signalized, this step is skipped.

The methodology includes a procedure for computing the proportion of vehicles that arrive during the effective green time for a phase serving a segment lane group (i.e., the lane groups "internal" to the segment). That procedure is described in this step. The platoon ratio (as described in Section 3 of Chapter 19, Signalized Intersections) should be used to compute this proportion for phases serving external lane groups.

If the upstream intersection is not signalized (or it is signalized but not coordinated with the downstream boundary intersection), the proportion arriving during the green is equal to the effective green-to-cycle-length ratio and this step is completed. This relationship implies that arrivals are effectively uniform during the cycle when averaged over the analysis period.

If the boundary intersections are coordinated, the remaining discussion in this step applies. The calculation of the proportion arriving during green is based on the signal timing of the upstream and downstream boundary intersections. However, if the signals are actuated, the resulting estimate of the proportion arriving during green typically has an effect on signal timing and capacity. In fact, the process is circular and requires an iterative sequence of calculations to arrive at a convergence solution in which all computed variables are in agreement with their initially assumed values. This process is illustrated in Exhibit 18-8. This exhibit indicates that the calculation of average phase duration is added to this process when the intersection is actuated.

Typically, there are three signalized traffic movements that depart the upstream boundary intersection at different times during the signal cycle. They are the cross-street right turn, the major-street through, and the cross-street left turn. Traffic may also enter the segment at various access point intersections. The signalized movements often enter the segment as a platoon, but this platoon disperses as the vehicles move down the segment.

A platoon dispersion model is used to predict the dispersed flow rate as a function of running time at any specified downstream location. The dispersed flow rates for the upstream intersection movement are combined with access point flow rates to predict an arrival flow profile at the downstream location. Exhibit 18-14 illustrates the predicted arrival flow profile at the stop line of the downstream intersection. This profile reflects the combination of the left-turn, through, and right-turn movements from the upstream intersection plus the turn movements at the access point intersection. The platoon dispersion model and the manner in which it is used to predict the dispersed flow rates for each of the individual movements are described in Section 3 of Chapter 30.

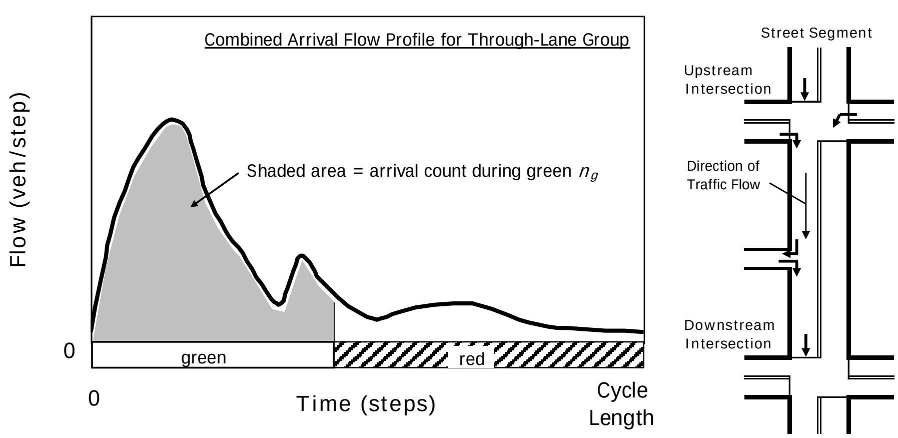


Exhibit 18-14
Use of an Arrival Flow Profile to Estimate the Volume Arriving During Green

The gray shaded area in Exhibit 18-14 represents the arrival count during green n_g. This count is computed by summing the flow rate for each time "step" (or interval) that occurs during the effective green period. The proportion of vehicles arriving during the effective green period for a specified lane group is computed with Equation 18-9.

Equation 18-9

$$P = \frac{n_g}{q_d\, C}$$

where

P = proportion of vehicles arriving during the green indication,

n_g = arrival count during green (veh),

q_d = arrival flow rate for downstream lane group (veh/s), and

C = cycle length (s).

Step 4: Determine Signal Phase Duration

This step applies to the downstream boundary intersection when the operation of a signalized urban street segment is evaluated. If the downstream boundary intersection is not signalized, this step is skipped.

If the downstream boundary intersection has pretimed signal control, the signal phase duration is an input value. If this intersection has some form of actuated control, the procedure described in Section 3 of Chapter 19 is used to estimate the average phase duration.

Steps 1 to 4 are repeated until the duration of each phase at each signalized intersection converges to its steady-state value. Convergence is indicated when the estimate of phase duration on two successive repetitions is the same.

Step 5: Determine Through Delay

The delay incurred by through vehicles as they exit the segment is the basis for travel time estimation. In this context, a through vehicle is a vehicle that enters and exits the segment as a through vehicle. The nature of the delay models used in this manual makes it difficult to separate the delay to through vehicles from the delay to nonthrough vehicles. However, these models can provide a reasonable estimate of through delay whenever the through movement is the dominant movement on the segment.

Through delay is the sum of two delay sources. One source, called control delay, is the delay due to the traffic control at the boundary intersection. The other, called geometric delay, is that due to the negotiation of intersection geometry, such as curvature.

Procedures for computing control delay are described in the following chapters of this manual:

- Signal control (Chapter 19 or 23),
- All-way STOP control (Chapter 21), and
- YIELD control at a roundabout intersection (Chapter 22).

The analyst should refer to the appropriate chapter for guidance in estimating the through control delay for the boundary intersection. If the through movement is uncontrolled at the boundary intersection, the through control delay is 0.0 s/veh.

The geometric delay for conventional three-leg or four-leg intersections (i.e., noncircular intersections) is considered to be negligible. In contrast, the geometric delay for a roundabout is not negligible. This delay can be estimated by using the procedure provided in Section 9 of Chapter 30.

If the segment is not in a coordinated system, the through delay estimate should be based on isolated operation. The methodologies in Chapters 19 to 22 can be used to provide this estimate.

If the segment is within a coordinated signal system, the methodology in Chapter 19 (for most signalized intersections) or Chapter 23 (for signalized ramp terminals and alternative intersections) is used to determine the through delay. The upstream filtering adjustment factor is used to account for the effect of the upstream signal on the variability in arrival volume at the downstream intersection. The equation for calculating this factor is described in Section 3 of Chapter 19.

If the through movement shares one or more lanes at a signalized boundary intersection, the through delay is computed by using Equation 18-10.

$$d_t = \frac{d_{th}\, v_t\, N_t + d_{sl}\, v_{sl}(1 - P_L) + d_{sr}\, v_{sr}(1 - P_R)}{v_{th}}$$

Equation 18-10

where

d_t = through delay (s/veh),

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

d_{th} = delay in exclusive-through lane group (s/veh),

v_t = demand flow rate in exclusive-through lane group (veh/h/ln),

N_t = number of lanes in exclusive-through lane group (ln),

d_{sl} = delay in shared left-turn and through lane group (s/veh),

v_{sl} = demand flow rate in shared left-turn and through lane group (veh/h),

d_{sr} = delay in shared right-turn and through lane group (s/veh),

v_{sr} = demand flow rate in shared right-turn and through lane group (veh/h),

P_L = proportion of left-turning vehicles in the shared lane (decimal), and

P_R = proportion of right-turning vehicles in the shared lane (decimal).

The procedure described in Section 2 of Chapter 31, Signalized Intersections: Supplemental, is used to estimate the variables shown in Equation 18-10.

Step 6: Determine Through Stop Rate

Through stop rate describes the stop rate of vehicles that enter and exit the segment as through vehicles. The nature of the stop rate models described in this step makes it difficult to separate the stops incurred by through vehicles from those incurred by nonthrough vehicles. However, the models can provide a reasonable estimate of through stop rate whenever the through movement is the dominant movement on the segment.

Stop rate is defined as the average number of full stops per vehicle. A *full stop* is defined to occur at a signalized intersection when a vehicle slows to zero (or a crawl speed, if in queue) as a consequence of the change in signal indication from green to red, but not necessarily in direct response to an observed red indication. A *full stop* is defined to occur at an unsignalized intersection when a vehicle slows to zero (or a crawl speed, if in queue) as a consequence of the control device used to regulate the approach. For example, if a vehicle is in an overflow queue and requires three signal cycles to clear the intersection, it is estimated to have three full stops (one stop for each cycle).

The stop rate for a STOP-controlled approach can be assumed to equal 1.0 stops/veh. The stop rate for an uncontrolled approach can be assumed to equal 0.0 stops/veh. The stop rate at a YIELD-controlled approach will vary with conflicting demand. It can be estimated (in stops per vehicle) as equal to the volume-to-capacity ratio of the through movement at the boundary intersection. This approach recognizes that YIELD control does not require drivers to come to a complete stop when there is no conflicting traffic.

The through stop rate at a signalized boundary intersection is computed by using Equation 18-11.

Equation 18-11

$$h = 3{,}600\left[\frac{N_f}{\min\left(1, \frac{v_{th}\,C}{N_{th}\,s\,g}\right) g\,s} + \frac{N_{th}\,Q_{2+3}}{v_{th}\,C}\right]$$

with

Equation 18-12

$$N_f = \frac{N_{f,t}\,N_t + N_{f,sl}(1 - P_L) + N_{f,sr}(1 - P_R)}{N_{th}}$$

Equation 18-13

$$s = \frac{s_t\,N_t + s_{sl}(1 - P_L) + s_{sr}(1 - P_R)}{N_{th}}$$

Equation 18-14

$$Q_{2+3} = \frac{\left(Q_{2,t} + Q_{3,t}\right)N_t + \left(Q_{2,sl} + Q_{3,sl}\right)(1 - P_L) + \left(Q_{2,sr} + Q_{3,sr}\right)(1 - P_R)}{N_{th}}$$

where

h = full stop rate (stops/veh),

N_f = number of fully stopped vehicles (veh/ln),

g = effective green time (s),

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

Q_{2+3} = back-of-queue size (veh/ln),

$N_{f,t}$ = number of fully stopped vehicles in exclusive-through lane group (veh/ln),

$N_{f,sl}$ = number of fully stopped vehicles in shared left-turn and through lane group (veh/ln),

$N_{f,sr}$ = number of fully stopped vehicles in shared right-turn and through lane group (veh/ln),

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

s_t = saturation flow rate in exclusive-through lane group (veh/h/ln),

s_{sl} = saturation flow rate in shared left-turn and through lane group with permitted operation (veh/h/ln),

s_{sr} = saturation flow rate in shared right-turn and through lane group with permitted operation (veh/h/ln),

$Q_{2,t}$ = second-term back-of-queue size for exclusive-through lane group (veh/ln),

$Q_{2,sl}$ = second-term back-of-queue size for shared left-turn and through lane group (veh/ln),

$Q_{2,sr}$ = second-term back-of-queue size for shared right-turn and through lane group (veh/ln),

$Q_{3,t}$ = third-term back-of-queue size for exclusive-through lane group (veh/ln),

$Q_{3,sl}$ = third-term back-of-queue size for shared left-turn and through lane group (veh/ln), and

$Q_{3,sr}$ = third-term back-of-queue size for shared right-turn and through lane group (veh/ln).

The procedure for computing N_f, Q_2, and Q_3 is provided in Section 4 of Chapter 31, Signalized Intersections: Supplemental.

The first term in Equation 18-11 represents the proportion of vehicles stopped once by the signal. For some of the more complex arrival–departure polygons that include left-turn movements operating with the permitted mode, the queue may dissipate at two or more points during the cycle. If this occurs, $N_{f,i}$ is computed for each of the i periods between queue dissipation points. The value of N_f then equals the sum of the $N_{f,i}$ values computed in this manner.

The second term in Equation 18-11 represents the additional stops that may occur during overflow (i.e., cycle failure) conditions. The contribution of this term becomes significant when the volume-to-capacity ratio exceeds about 0.8. The full stop rate typically varies from 0.4 stops/veh at low volume-to-capacity ratios to 2.0 stops/veh when the volume-to-capacity ratio is about 1.0.

Step 7: Determine Travel Speed

Equation 18-15 is used to compute the travel speed for the subject direction of travel along the segment.

Equation 18-15

$$S_{T,seg} = \frac{3{,}600\ L}{5{,}280\ (t_R + d_t)}$$

where

$S_{T,seg}$ = travel speed of through vehicles for the segment (mi/h),

L = segment length (ft),

t_R = segment running time (s), and

d_t = through delay (s/veh).

The delay used in Equation 18-15 is that incurred by the through lane group at the downstream boundary intersection.

Step 8: Determine Spatial Stop Rate

Spatial stop rate is the stop rate expressed in units of stops per mile. It provides an equitable means of comparing the performance of alternative street segments with differing lengths. Equation 18-16 is used to compute the spatial stop rate for the subject direction of travel along the segment.

Equation 18-16

$$H_{seg} = 5{,}280\,\frac{h + h_{\text{other}}}{L}$$

where

H_{seg} = spatial stop rate for the segment (stops/mi),

h = full stop rate (stops/veh),

h_{other} = full stop rate due to other sources (stops/veh), and

L = segment length (ft).

The full stop rate h used in Equation 18-16 is that incurred by the through lane group at the downstream boundary intersection. In some situations, stops may be incurred at midsegment locations due to pedestrian crosswalks, bus stops, or turns into access point approaches. If the full stop rate associated with these other stops can be estimated by the analyst, it can be included in the calculation by using the variable h_{other}.

Step 9: Determine LOS

LOS is determined separately for both directions of travel along the segment. Exhibit 18-1 lists the LOS thresholds established for this purpose. As indicated in this exhibit, LOS is defined by two performance measures. One measure is the travel speed for through vehicles. The second is the volume-to-capacity ratio for the through movement at the downstream boundary intersection.

The travel speed LOS threshold value is shown in Exhibit 18-1 to be dependent on the base free-flow speed. The base free-flow speed was computed in Step 2 and the travel speed was computed in Step 7.

The volume-to-capacity ratio for the through movement at the boundary intersection is computed as the through volume divided by the through-movement capacity. This capacity is an input variable to the methodology.

The LOS determined in this step applies to the overall segment for the subject direction of travel. This LOS describes conditions for the combined link and downstream boundary intersection. If desired, the methodologies in Chapters 19 to 23 can be used to determine the LOS for travel through just the downstream boundary intersection. The HCM does not include a methodology for describing the LOS for just the link portion of the segment.

LOS is probably more meaningful as an indicator of traffic performance along a facility rather than a single street segment. A procedure for estimating facility LOS is described in Chapter 16.

Step 10: Determine Automobile Traveler Perception Score

The automobile traveler perception score for urban street segments is provided as a useful performance measure. It indicates the traveler's perception of service quality. The score is computed with Equation 18-17 to Equation 18-22.

$$I_{a,seg} = 1 + P_{BCDEF} + P_{CDEF} + P_{DEF} + P_{EF} + P_F$$

Equation 18-17

with

$$P_{BCDEF} = \left(1 + e^{-1.1614 - 0.253\, H_{seg} + 0.3434\, P_{LTL,seg}}\right)^{-1}$$

Equation 18-18

$$P_{CDEF} = \left(1 + e^{0.6234 - 0.253\, H_{seg} + 0.3434\, P_{LTL,seg}}\right)^{-1}$$

Equation 18-19

$$P_{DEF} = \left(1 + e^{1.7389 - 0.253\, H_{seg} + 0.3434\, P_{LTL,seg}}\right)^{-1}$$

Equation 18-20

$$P_{EF} = \left(1 + e^{2.7047 - 0.253\, H_{seg} + 0.3434\, P_{LTL,seg}}\right)^{-1}$$

Equation 18-21

$$P_F = \left(1 + e^{3.8044 - 0.253\, H_{seg} + 0.3434\, P_{LTL,seg}}\right)^{-1}$$

Equation 18-22

where

$I_{a,seg}$ = automobile traveler perception score for segment;

P_{BCDEF} = probability that an individual will respond with a rating of B, C, D, E, or F;

P_{CDEF} = probability that an individual will respond with a rating of C, D, E, or F;

P_{DEF} = probability that an individual will respond with a rating of D, E, or F;

P_{EF} = probability that an individual will respond with a rating of E or F;

P_F = probability that an individual will respond with a rating of F; and

$P_{LTL,seg}$ = proportion of intersections with a left-turn lane (or bay) on the segment (decimal).

The derivation of Equation 18-17 is based on the assignment of scores to each letter rating, in which a score of "1" is assigned to the rating of A (denoting "best"), "2" is assigned to B, and so on. The survey results were used to calibrate a set of models that collectively predicts the probability that a traveler will assign various rating combinations for a specified spatial stop rate and proportion of intersections with left-turn lanes. The score obtained from Equation 18-17 represents the expected (or long-run average) score for the population of travelers.

The proportion of intersections with left-turn lanes equals the number of left-turn lanes (or bays) encountered while driving along the segment divided by the number of intersections encountered. The signalized boundary intersection is counted (if it exists). All unsignalized intersections of public roads are counted. Private driveway intersections are not counted unless they are signal controlled.

The score obtained from Equation 18-17 provides a useful indication of performance from the perspective of the traveler. Scores of 2.0 or less indicate the best perceived service, and values in excess of 5.0 indicate the worst perceived service. Although this score is closely tied to the concept of service quality, it is *not* used to determine LOS for the urban street segment.

4. PEDESTRIAN METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to pedestrians traveling along an urban street segment.

SCOPE OF THE METHODOLOGY

The overall scope of the four methodologies was provided in Section 2. This section identifies the additional conditions for which the pedestrian methodology is applicable.

- *Target travel modes.* The pedestrian methodology addresses travel by walking in the urban street right-of-way. It is not designed to evaluate the performance of other travel means (e.g., Segway, roller skates).
- *"Typical pedestrian" focus.* The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. The performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to the Americans with Disabilities Act requirements. For this reason, they should not be considered as a substitute for a formal compliance assessment of a pedestrian facility.

Spatial Limits

Travel Directions to Be Evaluated

Urban street performance from a pedestrian perspective is separately evaluated for each side of the street. *Unless otherwise stated, all variables identified in this section are specific to the subject side of the street.* If a sidewalk is not available for the subject side of the street, pedestrians are assumed to walk in the street on the subject side (even if there is a sidewalk on the other side).

The typical evaluation will focus on the performance of the segment (i.e., the link and boundary intersection combined). However, in some situations, an evaluation of just the link is appropriate. Each approach is discussed in this subsection.

Segment-Based Evaluation

For a segment-based evaluation, the pedestrian methodology considers the performance of the link and the boundary intersection. It is applied through a series of 10 steps culminating in the determination of the segment LOS.

A segment-based evaluation considers both pedestrian space and a pedestrian LOS score to determine segment LOS. It uses the worse of the LOS letters resulting from pedestrian space and the segment pedestrian LOS score to determine the overall segment pedestrian LOS. A segment-based evaluation is recommended for analyses that compare the LOS of multiple travel modes because each mode's segment LOS score and letter can be directly compared.

Pedestrian space reflects the level of crowding on the sidewalk. Pedestrian space typically only influences overall pedestrian LOS when pedestrian facilities are very narrow, pedestrian volumes are very high, or both. For example, with an effective sidewalk width of 4 ft, pedestrian volumes need to be in excess of 1,000 pedestrians per hour for the space-based pedestrian LOS to drop below LOS A. Pedestrian space is not applicable when the pedestrian facility does not exist.

The methodology supports the analysis of a segment with either signal-controlled or two-way STOP-controlled boundary intersections. Section 5 of Chapter 19 describes a methodology for evaluating signalized intersection performance from a pedestrian perspective. No methodology exists for evaluating two-way STOP-controlled intersection performance (with the cross street STOP controlled). However, it is reasoned that this type of control has negligible influence on pedestrian service along the segment. This edition of the HCM does not include a procedure for evaluating a segment's performance when the boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

Link-Based Evaluation

Only two of the 10 steps of the pedestrian methodology are used for link-based evaluation of pedestrian service. This approach is regularly used by local, regional, and state transportation agencies. It offers the advantage of being less data-intensive than the full 10-step methodology and produces results that are generally reflective of pedestrian perceptions of service along the roadway. It can be especially attractive when agencies are performing a networkwide evaluation for a large number of roadway links.

The analyst should recognize that the resulting link LOS does not consider some aspects of pedestrian travel along a segment (e.g., pedestrian space, crossing difficulty, or intersection service). For this reason, the LOS score for the link should not be aggregated to characterize facility performance. The analyst should also be aware that this approach precludes an integrated multimodal evaluation because it does not reflect all aspects of segment performance.

Performance Measures

Performance measures applicable to the pedestrian travel mode include pedestrian travel speed, pedestrian space, and pedestrian LOS score. The LOS score is an indication of the typical pedestrian's perception of the overall segment travel experience.

LOS is also considered a performance measure. It is useful for describing segment performance to elected officials, policy makers, administrators, or the public. LOS is based on pedestrian space and pedestrian LOS score.

"Pedestrian space" is the average amount of sidewalk area available to each pedestrian walking along the segment. A larger area is more desirable from the pedestrian perspective. Exhibit 18-15 provides a qualitative description of pedestrian space that can be used to evaluate sidewalk performance from a circulation-area perspective.

Exhibit 18-15
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p) Random Flow	Platoon Flow	Description
>60	>530	Ability to move in desired path, no need to alter movements
>40–60	>90–530	Occasional need to adjust path to avoid conflicts
>24–40	>40–90	Frequent need to adjust path to avoid conflicts
>15–24	>23–40	Speed and ability to pass slower pedestrians restricted
>8–15	>11–23	Speed restricted, very limited ability to pass slower pedestrians
≤8	≤11	Speed severely restricted, frequent contact with other users

The first two columns in Exhibit 18-15 indicate a sensitivity to flow condition. Random pedestrian flow is typical of most segments. Platoon flow is appropriate for shorter segments (e.g., in downtown areas) with signalized boundary intersections.

Limitations of the Methodology

This subsection identifies the known limitations of the pedestrian methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific street segment, the analyst should consider using alternative methods or tools for the evaluation.

The pedestrian methodology does not account for the effect of the following conditions on the quality of service provided to pedestrians:

- Segments bounded by an all-way STOP-controlled intersection, roundabout, or signalized interchange ramp terminal;
- Midsegment unsignalized crosswalks;
- Grades in excess of 2%;
- Pedestrian overcrossings for service across or along the segment;
- Points of high-volume pedestrian access to a sidewalk, such as a transit stop or a doorway from a large office building;
- Points where a high volume of vehicles cross the sidewalk, such as a parking garage entrance; and
- Presence of railroad crossings.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the pedestrian methodology. The required data are listed in Exhibit 18-16. They must be separately specified for each direction of travel on the segment and for each boundary intersection. The exhibit also lists default values that can be used if local data are not available (*2, 3*).

The data elements listed in Exhibit 18-16 do not include variables that are considered to represent calibration factors. A calibration factor typically has a relatively narrow range of reasonable values or has a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Exhibit 18-16
Required Input Data, Potential Data Sources, and Default Values for Pedestrian Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Traffic Characteristics	
Midsegment motorized vehicle flow rate[a] (veh/h)	Field data, past counts, forecasts	Must be provided
Midsegment pedestrian flow rate (p/h)	Field data, past counts	Must be provided
Proportion of on-street parking occupied (decimal)	Field data	0.50 (if parking lane present)
	Geometric Design	
Downstream intersection width[a] (ft)	Field data, aerial photo	Must be provided
Segment length[a] (ft)	Field data, aerial photo	Must be provided
Number of midsegment through lanes[a]	Field data, aerial photo	Must be provided
Outside through lane width (ft)	Field data, aerial photo	12 ft
Bicycle lane width (ft)	Field data, aerial photo	5.0 ft (if provided)
Paved outside shoulder width (ft)	Field data, aerial photo	Must be provided
Striped parking lane width (ft)	Field data, aerial photo	8.0 ft (if provided)
Curb presence (yes or no)	Field data, aerial photo	Must be provided
Sidewalk presence (yes or no)	Field data, aerial photo	Must be provided
Total walkway width (ft)	Field data, aerial photo	9.0 ft (business/office uses) 11.0 ft (residential/industrial uses)
Effective width of fixed objects (ft)	Field data	2.0 ft inside, 2.0 ft outside (business/office uses) 0.0 ft inside, 0.0 ft outside (residential/industrial uses)
Buffer width (ft)	Field data, aerial photo	0.0 ft (business/office uses) 6.0 ft (residential/industrial uses)
Spacing of objects in buffer (ft)	Field data, aerial photo	Must be provided
	Other Data	
Distance to nearest signal-controlled crossing (ft)	Field data, aerial photo	One-third the distance between signal-controlled crossings that bracket the segment
Legality of midsegment pedestrian crossing (legal or illegal)	Field data, local traffic laws	Must be provided
Proportion of sidewalk adjacent to window, building, or fence (decimal)	Field data	0.0 (non-CBD area) 0.5 building, 0.5 window (CBD)
	Performance Measures	
Motorized vehicle midsegment running speed[a] (mi/h)	HCM method output	Must be provided
Pedestrian delay at boundary intersection (s/p)	HCM method output	Must be provided
Pedestrian delay at midsegment signalized crosswalk (s/p)	HCM method output	20 s/p (if present)
Pedestrian delay at uncontrolled crossing (s/p)	HCM method output	Must be provided
Pedestrian LOS score for intersection (decimal)	HCM method output	Must be provided

Notes: CBD = central business district; p = person.
[a] Also used or calculated by the motorized vehicle methodology.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 18-16. These data describe the motorized vehicle and pedestrian traffic streams traveling along the segment during the analysis period. Midsegment flow rate is defined in a similarly titled section for the motorized vehicle methodology.

Pedestrian Flow Rate

The pedestrian flow rate is based on the count of pedestrians traveling along the outside of the subject segment during the analysis period. A separate count is taken for each direction of travel along the side of the segment. 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 for that side.

Proportion of On-Street Parking Occupied

This variable represents the proportion of the segment's right-hand curb line on which parked vehicles are present during the analysis period. It is computed as the sum of the curb line lengths occupied by parked vehicles divided by the link length. The use of pavement markings to delineate the parking lane should also be noted.

If parking is not allowed on the segment, the proportion equals 0.0. If parking is allowed along the segment but the spaces are not used during the analysis period, the proportion equals 0.0. If parking is allowed along the full length of the segment but only one-half of the spaces are occupied during the analysis period, the proportion equals 0.50.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 18-16. These data describe the geometric elements that influence pedestrian performance. All input data should be representative of the segment for its entire length. An average value should be used for each element that varies along the segment. Segment length and number of through lanes are defined in a similarly titled section for the motorized vehicle methodology.

Downstream Intersection Width

The intersection width applies to the downstream boundary intersection for a given direction of travel and represents the effective width of the cross street. On a two-way street, it is the distance between the stop (or yield) line for the two opposing segment through movements at the boundary intersection, as measured along the centerline of the segment. On a one-way street, it is the distance from the stop line to the far side of the most distant traffic lane on the cross street.

Width of Outside Through Lane, Bicycle Lane, Outside Shoulder, Parking Lane

The widths of several individual elements of the cross section are considered input data. These elements include the outside lane that serves motorized vehicles traveling along the segment, the bicycle lane adjacent to the outside lane (if used), paved outside shoulder, and striped parking lane.

The outside lane width does not include the width of the gutter. If curb and gutter are present, the width of the gutter is included in the shoulder width (i.e., shoulder width is measured to the curb face when a curb is present).

Curb Presence

The presence of a curb on the right side edge of the roadway is determined for each segment travel direction.

Presence of a Sidewalk

A sidewalk is a paved walkway that is provided at the side of the roadway. Pedestrians are assumed to walk in the street if a sidewalk is not present.

Total Walkway Width

Total walkway width is measured from the outside edge of the road pavement (or face of curb, if present) to the far edge of the sidewalk (as sometimes delineated by a building face or landscaping). It includes the width of any buffer (see below), if present. If this width varies along the segment, an average value is used. A paved shoulder is not included in this width measurement.

Effective Width of Fixed Objects

Two input variables are used to describe fixed objects along the walkway. One represents the effective width of objects along the inside of the sidewalk. These objects include light poles, traffic signs, planter boxes, and so forth. Typical widths for these objects are provided in Chapter 24, Off-Street Pedestrian and Bicycle Facilities. All objects along the sidewalk should be considered and an average value for the length of the sidewalk input to the methodology.

The second variable represents the effective width of objects along the outside of the sidewalk. It is determined in the same manner as was the first variable.

Buffer Width and Spacing of Objects in Buffer

The buffer width is the distance between the outside edge of the paved roadway (or face of curb, if present) and the near edge of the sidewalk. This element of the cross section is not designed for use by pedestrians or motorized vehicles. It may be unpaved or include various vertical objects that are continuous (e.g., barrier) or discontinuous (e.g., trees, bollards) to prevent pedestrian use. If vertical objects are in the buffer, the average spacing of objects that are 3 ft or more in height should also be recorded.

Other Data

This subsection describes the data listed in Exhibit 18-16 that are categorized as “other data.”

Distance to Nearest Signal-Controlled Crossing

This input variable is needed if there is an identifiable pedestrian path (*a*) that intersects the segment and continues beyond the segment and (*b*) on which most crossing pedestrians travel. This variable defines the distance pedestrians must travel along the segment should they divert from the path to cross the segment at the nearest signalized crossing. The crossing will typically be at a signalized intersection. However, it may also be at a signalized crosswalk provided at a midsegment location. If the crossing is at a signalized intersection, it will likely occur in the crosswalk on the side of the intersection that is nearest to the segment. Occasionally, it will be on the far side of the intersection because the near-side crosswalk is closed (or a crossing at this location is otherwise prohibited). This distance is measured along one side of the subject segment; the methodology accounts for the return distance once the pedestrian arrives at the other side of the segment.

Legality of Midsegment Pedestrian Crossing

This input indicates whether a pedestrian can cross the segment at any point along its length, regardless of location. If making this crossing at any point is illegal, the pedestrian is assumed to be required to divert to the nearest signalized intersection to cross the segment.

Proportion of Sidewalk Adjacent to Window, Building, or Fence

Three proportions are input for a sidewalk. One proportion represents the length of sidewalk adjacent to a fence or low wall divided by the length of the link. The second represents the length of the sidewalk adjacent to a building face divided by the length of the link. The final proportion represents the length of the sidewalk adjacent to a window display divided by the length of the link.

Performance Measures

This subsection describes the data listed in Exhibit 18-16 that are categorized as "performance measures."

Motorized Vehicle Running Speed

The motorized vehicle running speed is based on the segment running time obtained from the motorized vehicle methodology. The running speed is equal to the segment length divided by the segment running time.

Pedestrian Delay

Three pedestrian delay variables are needed. The first is the delay to pedestrians who travel through the boundary intersection along a path that is parallel to the segment centerline. The pedestrian movement of interest is traveling on the subject side of the street and heading in a direction that is "with" or "against" the motorized traffic stream. For a two-way STOP-controlled boundary intersection, this delay is reasoned to be negligible. For a signal-controlled boundary intersection, the procedure described in Section 3 of Chapter 19 is used to compute this delay.

The second delay variable describes the delay incurred by pedestrians who cross the subject segment at the *nearest* signal-controlled crossing. If the nearest crossing is at a signalized intersection, the procedure described in Section 3 of Chapter 19 is used to compute this delay. If the nearest crossing is at a midsegment signalized crosswalk, this delay should equal the pedestrian's average wait for service after the pedestrian push button is pressed. This wait will depend on the signal settings and could range from 5 to 25 seconds per pedestrian (s/p).

The third delay variable needed is the pedestrian waiting delay. This delay is incurred when pedestrians wait at an uncontrolled crossing location. If this type of crossing is legal, the pedestrian waiting delay is determined by using the procedure in Chapter 20, Two-Way STOP-Controlled Intersections. If it is illegal, the pedestrian waiting delay does not need to be calculated.

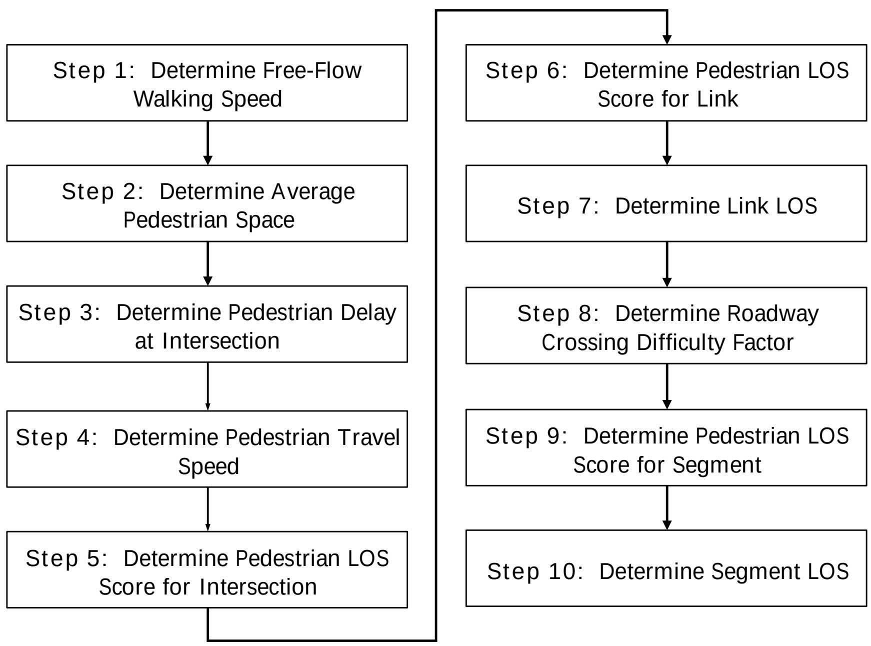
Pedestrian LOS Score for Intersection

The pedestrian LOS score for the signalized intersection is obtained from the pedestrian methodology in Chapter 19.

OVERVIEW OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of an urban street segment in terms of its service to pedestrians. The methodology consists of 10 calculation steps. These steps are illustrated in Exhibit 18-17. All 10 steps are completed for a typical segment-based evaluation. Only Steps 6 and 7 are needed for a link-based evaluation.

Exhibit 18-17
Pedestrian Methodology for Urban Street Segments



A methodology for evaluating off-street pedestrian facilities is provided in Chapter 24, Off-Street Pedestrian and Bicycle Facilities.

COMPUTATIONAL STEPS

Step 1: Determine Free-Flow Walking Speed

The *average* free-flow pedestrian walking speed S_{pf} is needed for the evaluation of urban street segment performance from a pedestrian perspective. This speed should reflect conditions in which there are negligible pedestrian-to-pedestrian conflicts and negligible adjustments in a pedestrian's desired walking path to avoid other pedestrians.

Research indicates that walking speed is influenced by pedestrian age and sidewalk grade (*4*). If 0% to 20% of pedestrians traveling along the subject segment are elderly (i.e., 65 years of age or older), an average free-flow walking speed of 4.4 ft/s is recommended for segment evaluation. If more than 20% of pedestrians are elderly, an average free-flow walking speed of 3.3 ft/s is recommended. In addition, an upgrade of 10% or greater reduces walking speed by 0.3 ft/s.

Step 2: Determine Average Pedestrian Space

Pedestrians are sensitive to the amount of space separating them from other pedestrians and obstacles as they walk along a sidewalk. Average pedestrian space is an indicator of segment performance for travel in a sidewalk. It depends on the effective sidewalk width, pedestrian flow rate, and walking speed. This step is not applicable when the sidewalk does not exist.

A. Compute Effective Sidewalk Width

The effective sidewalk width equals the total walkway width less the effective width of fixed objects located on the sidewalk and less any shy distance associated with the adjacent street or a vertical obstruction. Fixed objects can be continuous (e.g., a fence or a building face) or discontinuous (e.g., trees, poles, or benches).

The effective sidewalk width is an average value for the length of the link. It is computed by using Equation 18-23 to Equation 18-27.

$$W_E = W_T - W_{O,i} - W_{O,o} - W_{s,i} - W_{s,o} \geq 0.0$$

Equation 18-23

with

$$W_{s,i} = \max\,(W_{buf}, 1.5)$$

Equation 18-24

$$W_{s,o} = 3.0\, p_{\text{window}} + 2.0\, p_{\text{building}} + 1.5\, p_{\text{fence}}$$

Equation 18-25

$$W_{O,i} = w_{O,i} - W_{s,i} \;\geq 0.0$$

Equation 18-26

$$W_{O,o} = w_{O,o} - W_{s,o} \;\geq 0.0$$

Equation 18-27

where

W_E = effective sidewalk width (ft),

W_T = total walkway width (ft),

$W_{O,i}$ = adjusted fixed-object effective width on inside of sidewalk (ft),

$W_{O,o}$ = adjusted fixed-object effective width on outside of sidewalk (ft),

$W_{s,i}$ = shy distance on inside (curb side) of sidewalk (ft),

$W_{s,o}$ = shy distance on outside of sidewalk (ft),

W_{buf} = buffer width between roadway and sidewalk (ft),

p_{window} = proportion of sidewalk length adjacent to a window display (decimal),

p_{building} = proportion of sidewalk length adjacent to a building face (decimal),

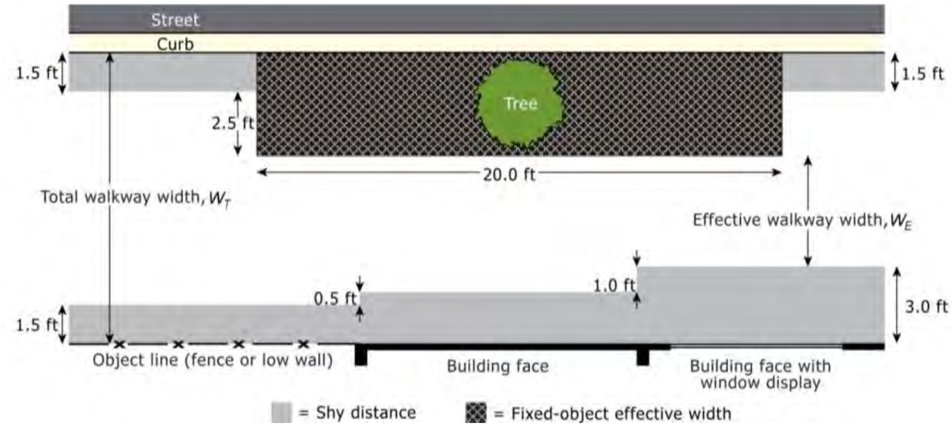
p_{fence} = proportion of sidewalk length adjacent to a fence or low wall (decimal),

$w_{O,i}$ = effective width of fixed objects on inside of sidewalk (ft), and

$w_{O,o}$ = effective width of fixed objects on outside of sidewalk (ft).

The relationship between the variables in these equations is illustrated in Exhibit 18-18.

Exhibit 18-18
Width Adjustments for Fixed Objects



The variables W_T, W_{buf}, p_{window}, p_{building}, p_{fence}, $w_{O,i}$, and $w_{O,o}$ are input variables. They represent average, or typical, values for the length of the sidewalk. Chapter 24, Off-Street Pedestrian and Bicycle Facilities, provides guidance for estimating the effective width of many common fixed objects.

Typical shy distances are shown in Exhibit 18-18. Shy distance on the inside (curb side) of the sidewalk is measured from the outside edge of the paved roadway (or face of curb, if present). It is generally considered to equal 1.5 ft. Shy distance on the outside of the sidewalk is 1.5 ft if a fence or a low wall is present, 2.0 ft if a building is present, 3.0 ft if a window display is present, and 0.0 ft otherwise.

B. Compute Pedestrian Flow Rate per Unit Width

The pedestrian flow per unit width of sidewalk is computed by using Equation 18-28 for the subject sidewalk. The variable v_{ped} is an input variable.

Equation 18-28

$$v_p = \frac{v_{ped}}{60\,W_E}$$

where

v_p = pedestrian flow per unit width (p/ft/min),

v_{ped} = pedestrian flow rate in the subject sidewalk (walking in both directions) (p/h), and

W_E = effective sidewalk width (ft).

C. Compute Average Walking Speed

The average walking speed S_p is computed by using Equation 18-29. This equation is derived from the relationship between flow rate and average walking speed described in Exhibit 24-1 of Chapter 24.

Equation 18-29

$$S_p = \left(1 - 0.00078\,v_p^2\right) S_{pf} \geq 0.5\,S_{pf}$$

where S_p is the pedestrian walking speed (ft/s), S_{pf} is the free-flow pedestrian walking speed (ft/s), and v_p is the pedestrian flow per unit width (p/ft/min).

D. Compute Pedestrian Space

Finally, Equation 18-30 is used to compute average pedestrian space.

$$A_p = 60\frac{S_p}{v_p}$$

Equation 18-30

where A_p is the pedestrian space (ft²/p) and all other variables are as previously defined.

The pedestrian space obtained from Equation 18-30 can be compared with the ranges provided in Exhibit 18-15 to make some judgments about the performance of the subject intersection corner.

Step 3: Determine Pedestrian Delay at Intersection

Pedestrian delay at three locations along the segment is determined in this step. Each of these delays is an input variable for the methodology and is described in the previous subsection titled Required Data and Sources.

The first delay variable d_{pp} represents the delay incurred by pedestrians who travel through the boundary intersection along a path that is parallel to the segment centerline. The second delay variable d_{pc} represents the delay incurred by pedestrians who cross the segment at the nearest signal-controlled crossing. The third delay variable d_{pw} represents the delay incurred by pedestrians waiting for a gap to cross the segment at an uncontrolled location.

Step 4: Determine Pedestrian Travel Speed

Pedestrian travel speed represents an aggregate measure of speed along the segment. It combines the delay incurred at the downstream boundary intersection and the time required to walk the length of the segment. Thus, it is typically slower than the average walking speed. The pedestrian travel speed is computed by using Equation 18-31.

$$S_{Tp,seg} = \frac{L}{\frac{L}{S_p} + d_{pp}}$$

Equation 18-31

where

$S_{Tp,seg}$ = travel speed of through pedestrians for the segment (ft/s),

L = segment length (ft),

S_p = pedestrian walking speed (ft/s), and

d_{pp} = pedestrian delay incurred in walking parallel to the segment (s/p).

In general, a travel speed of 4.0 ft/s or more is considered desirable and a speed of 2.0 ft/s or less is considered undesirable.

Step 5: Determine Pedestrian LOS Score for Intersection

The pedestrian LOS score for the boundary intersection $I_{p,int}$ is determined in this step. If the boundary intersection is signalized, the pedestrian methodology described in Chapter 19 is used for this determination. If the boundary intersection is two-way STOP controlled, the score is equal to 0.0.

Step 6: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link $I_{p,\text{link}}$ is calculated with Equation 18-32.

Equation 18-32

$$I_{p,\text{link}} = 6.0468 + F_w + F_v + F_S$$

with

Equation 18-33

$$F_w = -1.2276 \ln\left(W_v + 0.5\, W_l + 50\, p_{pk} + W_{buf} f_b + W_{aA} f_{sw}\right)$$

Equation 18-34

$$F_v = 0.0091 \frac{v_m}{4\, N_{th}}$$

Equation 18-35

$$F_S = 4\left(\frac{S_R}{100}\right)^2$$

where

$I_{p,\text{link}}$ = pedestrian LOS score for link;

F_w = cross-section adjustment factor;

F_v = motorized vehicle volume adjustment factor;

F_S = motorized vehicle speed adjustment factor;

$\ln(x)$ = natural log of x;

W_v = effective total width of outside through lane, bicycle lane, and shoulder as a function of traffic volume (see Exhibit 18-19) (ft);

W_l = total width of shoulder, bicycle lane, and parking lane (see Exhibit 18-19) (ft);

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

W_{buf} = buffer width between roadway and available sidewalk (= 0.0 if sidewalk does not exist) (ft);

f_b = buffer area coefficient = 5.37 for any continuous barrier at least 3 ft high that is located between the sidewalk and the outside edge of roadway; otherwise use 1.0;

W_A = available sidewalk width = 0.0 if sidewalk does not exist or $W_T - W_{buf}$ if sidewalk exists (ft);

W_T = total walkway width (ft);

W_{aA} = adjusted available sidewalk width = min(W_A , 10) (ft);

f_{sw} = sidewalk width coefficient = 6.0 – 0.3 W_{aA};

v_m = midsegment demand flow rate (direction nearest to the subject sidewalk) (veh/h);

N_{th} = number of through lanes on the segment in the subject direction of travel (ln); and

S_R = motorized vehicle running speed = (3,600 L)/(5,280 t_R) (mi/h).

The value used for several of the variables in Equation 18-33 to Equation 18-35 is dependent on various conditions. These conditions are identified in Column 1 of Exhibit 18-19. If the condition is satisfied, the equation in Column 2 is used to compute the variable value. If it is not satisfied, the equation in Column 3 is used. The equations in the first two rows are considered in sequence to determine the effective width of the outside lane and shoulder W_v.

Exhibit 18-19
Variables for Pedestrian LOS Score for Link

Condition	Variable When Condition Is Satisfied	Variable When Condition Is Not Satisfied
$v_m > 160$ veh/h or $W_A > 0$ ft	$W_v = W_{ol} + W_{bl} + W_{os}^* + W_{pk}$	$W_v = (W_{ol} + W_{bl} + W_{os}^* + W_{pk}) \times (2 - 0.005\ v_m)$
$p_{pk} > 0.25$ or $W_{bl} + W_{os}^* + W_{pk} \leq 10$	$W_l = W_{bl} + W_{os}^* + W_{pk}$	$W_l = 10$

Notes: W_{ol} = width of the outside through lane (ft);
W_{os}^* = adjusted width of paved outside shoulder; if curb is present $W_{os}^* = W_{os} - 1.5 \geq 0.0$, otherwise $W_{os}^* = W_{os}$ (ft);
W_{os} = width of paved outside shoulder (ft);
W_{bl} = width of the bicycle lane = 0.0 if bicycle lane not provided (ft); and
W_{pk} = width of striped parking lane (ft).

The buffer width coefficient determination is based on the presence of a continuous barrier in the buffer. In making this determination, repetitive vertical objects (e.g., trees or bollards) are considered to represent a continuous barrier if they are at least 3 ft high and have an average spacing of 20 ft or less. For example, the sidewalk shown in Exhibit 18-18 does not have a continuous buffer because the street trees adjacent to the curb are spaced at more than 20 ft.

The pedestrian LOS score is sensitive to the separation between pedestrians and moving vehicles and to the speed and volume of these vehicles. Physical barriers and parked cars between moving vehicles and pedestrians effectively increase the separation distance and the perceived quality of service. Higher vehicle speeds or volumes lower the perceived quality of service.

If the sidewalk is not continuous for the length of the segment, the segment should be subdivided into subsegments and each subsegment separately evaluated. For this application, a subsegment is defined to begin or end at each break in the sidewalk. Each subsegment is then separately evaluated by using Equation 18-32. Each equation variable is uniquely quantified to represent the subsegment to which it applies. The buffer width and the effective sidewalk width are each set to 0.0 ft for any subsegment without a sidewalk. The pedestrian LOS score $I_{p,\text{link}}$ is then computed as a weighted average of the subsegment scores, where the weight assigned to each score equals the portion of the segment length represented by the corresponding subsegment.

The motorized vehicle running speed is computed by using the motorized vehicle methodology, as described in Section 3.

Step 7: Determine Link LOS

The pedestrian LOS for the link is determined by using the pedestrian LOS score from Step 6. This score is compared with the link-based pedestrian LOS thresholds on the right side of Exhibit 18-2 to determine the LOS for the specified direction of travel along the subject link.

Step 8: Determine Roadway Crossing Difficulty Factor

The pedestrian roadway crossing difficulty factor measures the difficulty of crossing the street between boundary intersections. Segment performance from a pedestrian perspective is reduced if the crossing is perceived to be difficult.

The roadway crossing difficulty factor is based on the delay incurred by a pedestrian who crosses the subject segment. One crossing option the pedestrian may consider is to alter his or her travel path by diverting to the nearest signal-controlled crossing. This crossing location may be a midsegment signalized crosswalk or a signalized intersection.

A second crossing option is to continue on the original travel path by completing a midsegment crossing at an uncontrolled location. If this type of crossing is legal along the subject segment, the pedestrian crosses when there is an acceptable gap in the motorized vehicle stream.

Each of these two crossing options is considered in this step, with the option requiring the least perceived delay used as the basis for computing the pedestrian roadway crossing difficulty factor. The time to walk across the roadway is common to both options and therefore is not included in the delay estimate for either option.

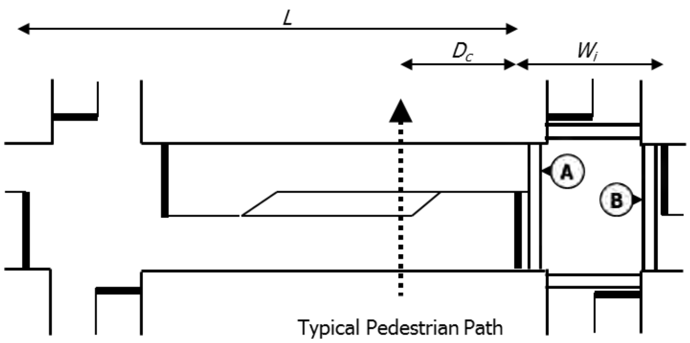
The delay incurred as a consequence of diverting to the nearest signal-controlled crossing is computed first. It includes the delay involved in walking to and from the midsegment crossing point to the nearest signal-controlled crossing and the delay waiting to cross at the signal. Hence, calculation of this delay requires knowledge of the distance to the nearest signalized crossing and its signal timing.

The distance to the nearest crossing location D_c is based on one of two approaches. The first approach is used if there is an identifiable pedestrian path (*a*) that intersects the segment and continues on beyond the segment and (*b*) on which most crossing pedestrians travel. The location of this path is shown for two cases in Exhibit 18-20. Exhibit 18-20(a) illustrates the distance D_c when the pedestrian diverts to the nearest signalized intersection. This distance is measured from the crossing location to the signalized intersection.

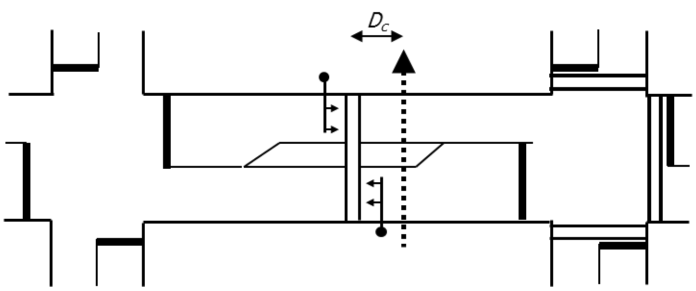
Exhibit 18-20(b) illustrates the distance D_c when a signalized crosswalk is provided at a midsegment location. In this situation, the distance is measured from the pedestrian crossing location to the location of the signalized crosswalk. In either case, the distance D_c is an input value provided by the analyst.

The second approach is used if crossings occur somewhat uniformly along the length of the segment. In this situation the distance D_c can be assumed to equal one-third of the distance between the nearest signal-controlled crossings that bracket the subject segment.

Exhibit 18-20
Diversion Distance Components



(a) Divert to Nearest Boundary Intersection



(b) Divert to Midsegment Signalized Crosswalk

The diversion distance to the nearest crossing is computed with Equation 18-36.

Equation 18-36

$$D_d = 2\ D_c$$

where

D_d = diversion distance (ft), and

D_c = distance to nearest signal-controlled crossing (ft).

If the nearest crossing location is at the signalized intersection and the crossing is at Location A in Exhibit 18-20(a), Equation 18-36 applies directly. If the nearest crossing location is at the signalized intersection but the crossing is at Location B, the distance obtained from Equation 18-36 should be increased by adding two increments of the intersection width W_i.

The delay incurred due to diversion is calculated by using Equation 18-37.

Equation 18-37

$$d_{pd,LOS} = 0.084 \frac{2D_d}{S_p} + d_{pc}$$

where

$d_{pd,LOS}$ = LOS-based pedestrian-perceived diversion delay (s/p),

D_d = diversion distance (ft),

S_p = pedestrian walking speed (ft/s), and

d_{pc} = pedestrian delay incurred in crossing the segment at the nearest signal-controlled crossing (s/p), determined previously in Step 3.

The second crossing option is to cross the roadway at an uncontrolled midsegment location. The average delay to do so d_{pw} can be calculated using the uncontrolled pedestrian crossing procedure in Chapter 20. However, if a midsegment crossing is illegal, then only diversion delay is used for the remainder of this step.

The LOS-based pedestrian-perceived diversion delay and the pedestrian waiting delay are each converted to an equivalent LOS score by using the thresholds listed in Exhibit 18-20A. Specifically, $d_{pd,LOS}$ is used with Exhibit 18-20A to determine the LOS score for diversion delay I_{pd}. Similarly, d_{pw} is used with Exhibit 18-20A to determine the LOS score for waiting delay I_{pw}. When either delay value is between the range limits shown in the table, interpolation is used to estimate the corresponding LOS score.

Exhibit 18-20A
LOS Scores Associated with Ranges of Midblock Pedestrian Delay

Delay Range (s)	Equivalent LOS Score Range
0 and ≤10	0 and ≤1.5
>10 and ≤20	>1.5 and ≤2.5
>20 and ≤30	>2.5 and ≤3.5
>30 and ≤40	>3.5 and ≤4.5
>40 and ≤60	>4.5 and ≤5.5
>60 and ≤70	>5.5 and ≤6.0
>70	6.0

Finally, the midsegment crossing LOS score is computed using the following equation.

Equation 18-38

$$I_{p,mx} = \min\left[I_{pw}, I_{pd}, 6\right]$$

where

$I_{p,mx}$ = pedestrian LOS score for midsegment crossing (A = 1, B = 2, ..., F = 6);

I_{pw} = LOS score for pedestrian waiting delay (based on Exhibit 18-20A); and

I_{pd} = LOS score for pedestrian diversion delay (based on Exhibit 18-20A).

Step 9: Determine Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is computed with Equation 18-39:

Equation 18-39

$$I_{p,seg} = \left[\frac{\left(I_{p,\text{link}}\left[1 - p_{mx}\right] + I_{p,mx}\, p_{mx}\right)^3 L/S_p + \left(I_{p,int}\right)^3 d_{pp}}{L/S_p + d_{pp}}\right]^{1/3}$$

where $I_{p,seg}$ is the pedestrian LOS score for the segment, p_{mx} is the proportion of pedestrian demand that desires to cross at a midsegment location (default: 0.35), and all other variables are as previously defined.

The segment LOS score is a weighted average of three separate LOS scores. As a result, it is likely to be less sensitive to a change in any one of the three separate LOS scores. In other words, the segment LOS score can mask important factors that are influencing link, intersection, or midsegment crossing LOS in isolation. For this reason, the analyst should consider the link and intersection LOS scores individually to ensure that all factors influencing system performance are fully considered. This recommendation is extended to the analyst's consideration of the midsegment crossing LOS score.

Step 10: Determine Segment LOS

The pedestrian LOS for the segment is determined by using the pedestrian LOS score from Step 9 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit 18-2 to determine the LOS for the specified direction of travel along the subject segment. If a sidewalk does not exist and pedestrians are relegated to walking in the street, LOS is determined by using Exhibit 18-3 because the pedestrian space concept does not apply.

5. BICYCLE METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to bicyclists traveling along an urban street segment.

SCOPE OF THE METHODOLOGY

The overall scope of the four methodologies was provided in Section 2. This section identifies the additional conditions for which the bicycle methodology is applicable.

- *Target travel modes.* The bicycle methodology addresses travel by bicycle in the urban street right-of-way. It is not designed to evaluate the performance of other travel means (e.g., motorized bicycle, rickshaw).
- *Shared or exclusive lanes.* The bicycle methodology can be used to evaluate the service provided to bicyclists when they share a lane with motorized vehicles or when they travel in an exclusive bicycle lane.

Spatial Limits

Travel Directions to Be Evaluated

Urban street segment performance from a bicyclist perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.* The bicycle is assumed to travel in the street (possibly in a bicycle lane) and in the same direction as adjacent motorized vehicles.

The typical evaluation will focus on the performance of the segment (i.e., the link and boundary intersection combined). However, in some situations, an evaluation of just the link is appropriate. Each approach is discussed in this subsection.

Segment-Based Evaluation

For a segment-based evaluation, the bicycle methodology considers the performance of the link and the boundary intersection. It is applied through a series of eight steps that culminate in the determination of the segment LOS.

The methodology supports the analysis of a segment with either signal-controlled or two-way STOP-controlled boundary intersections. Chapter 19 describes a methodology for evaluating signalized intersection performance from a bicyclist perspective. No methodology exists for evaluating two-way STOP-controlled intersection performance (with the cross street STOP controlled). However, the influence of this type of control is incorporated in the methodology for evaluating segment performance. This edition of the HCM does not include a procedure for evaluating a segment's performance when the boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

Link-Based Evaluation

Only two of the eight steps of the bicycle methodology are used for link-based evaluation of bicycle service. This approach is regularly used by local, regional, and state transportation agencies. It offers the advantage of being less data-intensive than the full eight-step methodology and produces results that are generally reflective of bicyclist perceptions of service along the roadway. It can be especially attractive when agencies are performing a networkwide evaluation for a large number of roadway links.

The analyst should recognize that the resulting link LOS does not consider some aspects of bicycle travel along a segment (e.g., intersection service). For this reason, the LOS score for the link should not be aggregated for the purpose of characterizing facility performance. The analyst should also be aware that this approach precludes an integrated multimodal evaluation because it does not reflect all aspects of segment performance.

Performance Measures

Performance measures applicable to the bicycle travel mode include bicycle travel speed and bicycle LOS score. The LOS score is an indication of the typical bicyclist's perception of the overall segment travel experience.

LOS is also considered a performance measure. It is useful for describing segment performance to elected officials, policy makers, administrators, or the public. LOS is based on the bicyclist LOS score.

Limitations of the Methodology

This subsection identifies the known limitations of the bicycle methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific street segment, the analyst should consider using alternative methods or tools for the evaluation.

The bicycle methodology does not account for the effect of the following conditions on the quality of service provided to bicyclists:

- Segments bounded by an all-way STOP-controlled intersection, roundabout, or signalized interchange ramp terminal;
- Grades in excess of 2%; and
- Presence of railroad crossings.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the bicycle methodology. The required data are listed in Exhibit 18-21. They must be separately specified for each direction of travel on the segment and for each boundary intersection. The exhibit also lists default values that can be used if local data are not available (*2*, *3*).

Exhibit 18-21
Required Input Data, Potential Data Sources, and Default Values for Bicycle Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Traffic Characteristics	
Midsegment motorized vehicle flow rate[a] (veh/h)	Field data, past counts, forecasts	Must be provided
Heavy vehicle percentage (%)	Field data, past counts	Must be provided
Proportion of on-street parking occupied (decimal)	Field data	0.50 (if parking lane present)
	Geometric Design	
Segment length[a] (ft)	Field data, aerial photo	Must be provided
Number of midsegment through lanes[a]	Field data, aerial photo	Must be provided
Outside through lane width (ft)	Field data, aerial photo	12 ft
Bicycle lane width (ft)	Field data, aerial photo	5.0 ft (if provided)
Paved outside shoulder width (ft)	Field data, aerial photo	Must be provided
Striped parking lane width (ft)	Field data, aerial photo	Must be provided
Median type (divided or undivided)	Field data, aerial photo	Must be provided
Curb presence (yes or no)	Field data	Must be provided
Number of access point approaches	Field data, aerial photo	See discussion in text
	Other Data	
Pavement condition[b] (FHWA 5-point scale)	Field data, pavement condition inventory	3.5 (good)
	Performance Measures	
Motorized vehicle midsegment running speed[a] (mi/h)	HCM method output	Must be provided
Bicycle delay at boundary int. (s/bicycle)	HCM method output	Must be provided
Bicycle LOS score at boundary int. (decimal)	HCM method output	Must be provided

Notes: FHWA = Federal Highway Administration; int. = intersection.
Bold italic indicates high sensitivity (±2 LOS letters) of LOS to the choice of default value.
Bold indicates moderate sensitivity (±1 LOS letter) of LOS to the choice of default value.
[a] Also used or calculated by the motorized vehicle methodology.
[b] Sensitivity reflects pavement conditions 2–5. Very poor pavement (i.e., 1) typically results in LOS F, regardless of other input values.

The data elements listed in Exhibit 18-21 do not include variables that are considered to represent calibration factors. A calibration factor typically has a relatively narrow range of reasonable values or has 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.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 18-21. These data describe the motorized vehicle and bicycle traffic streams traveling along the segment during the analysis period. Midsegment flow rate is defined in Section 3 for the motorized vehicle mode. The "proportion of on-street parking occupied" is defined in Section 4 for the pedestrian mode.

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* is 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 the same location on the segment as represented by the midsegment flow rate.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 18-21. These data describe the geometric elements that influence bicycle performance. All input data should be representative of the segment for its entire length. An average value should be used for each element that varies along the segment.

Most of the geometric design input data are defined in previous sections. Segment length, number of through lanes, and number of access point approaches are defined in Section 3 for the motorized vehicle mode. The following variables are defined in Section 4 for the pedestrian mode: width of outside through lane, width of bicycle lane, width of paved outside shoulder, width of striped parking lane, and curb presence.

Median type is designated as "undivided" or "divided." A street is indicated to have a divided median type if it has a nonrestrictive median (e.g., two-way left-turn lane) or restrictive (e.g., raised curb) median; otherwise, it is undivided.

Other Data

This subsection describes the data listed in Exhibit 18-21 that are categorized as "other data."

The *pavement condition rating* describes the road surface in terms of ride quality and surface defects. It is based on the present serviceability rating, a subjective rating system based on a scale of 0 to 5 (*5*). Exhibit 18-22 provides a description of pavement conditions associated with various ratings.

Exhibit 18-22
Pavement Condition Rating

Pavement Condition Rating	Pavement Description	Motorized Vehicle Ride Quality and Traffic Speed
4.0 to 5.0	New or nearly new superior pavement. Free of cracks and patches.	Good ride
3.0 to 4.0	Flexible pavements may begin to show evidence of rutting and fine cracks. Rigid pavements may begin to show evidence of minor cracking.	Good ride
2.0 to 3.0	Flexible pavements may show rutting and extensive patching. Rigid pavements may have a few joint fractures, faulting, or cracking.	Acceptable ride for low-speed traffic but barely tolerable for high-speed traffic
1.0 to 2.0	Distress occurs over 50% or more of the surface. Flexible pavement may have large potholes and deep cracks. Rigid pavement distress includes joint spalling, patching, and cracking.	Pavement deterioration affects the speed of free-flow traffic; ride quality not acceptable
0.0 to 1.0	Distress occurs over 75% or more of the surface. Large potholes and deep cracks exist.	Passable only at reduced speed and considerable rider discomfort

Performance Measures

This subsection describes the data listed in Exhibit 18-21 that are categorized as "performance measures."

Motorized Vehicle Running Speed

The motorized vehicle running speed is based on the segment running time obtained from the motorized vehicle methodology. The running speed is equal to the segment length divided by the segment running time.

Bicycle Delay

Bicycle delay is the delay to bicyclists who travel through the boundary intersection along a path that is parallel to the segment centerline. The bicycle movement of interest is traveling on the subject side of the street and heading in the same direction as motorized vehicles. For a two-way STOP-controlled boundary intersection, this delay is reasoned to be negligible. For a signal-controlled boundary intersection, the procedure described in Section 3 of Chapter 19 is used to compute this delay.

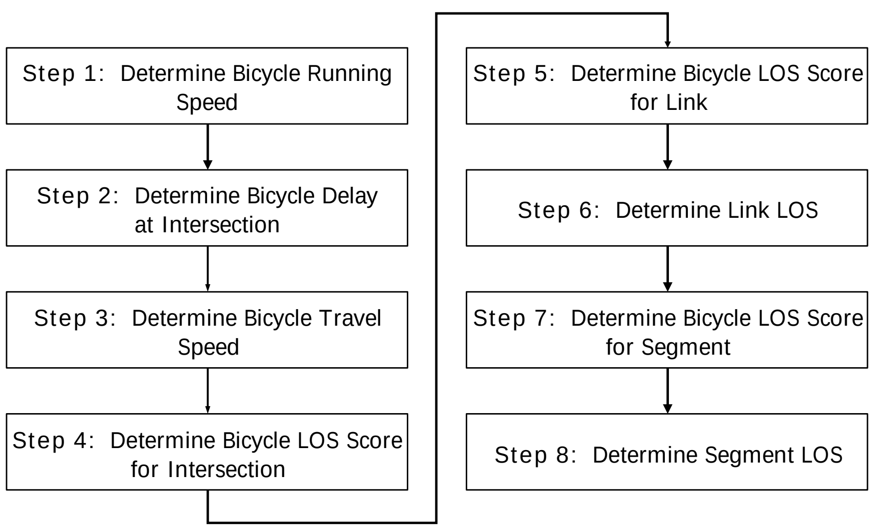
Bicycle LOS Score for Intersection

The bicycle LOS score for the signalized intersection is obtained from the bicycle methodology in Chapter 19.

OVERVIEW OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of an urban street segment in terms of its service to bicyclists. The methodology consists of eight calculation steps. These steps are illustrated in Exhibit 18-23. All eight steps are completed for the typical segment-based evaluation. Only Steps 5 and 6 are needed for a link-based evaluation.

Exhibit 18-23
Bicycle Methodology for Urban Street Segments



A methodology for evaluating off-street bicycle facilities is provided in Chapter 24, Off-Street Pedestrian and Bicycle Facilities.

COMPUTATIONAL STEPS

Step 1: Determine Bicycle Running Speed

An estimate of the *average* bicycle running speed S_b is determined in this step. The best basis for this estimate is a field measurement of midsegment bicycle speed on representative streets in the vicinity of the subject street. In the absence of this information, the average running speed of bicycles is recommended to be taken as 15 mi/h between signalized intersections (*6*). Many factors might affect bicycle speed, including adjacent motor vehicle traffic, adjacent on-street parking activity, commercial and residential driveways, lateral obstructions, and significant grades. To date, research is not available to make any specific recommendations as to the effect of these factors on speed.

Step 2: Determine Bicycle Delay at Intersection

Bicycle delay at the boundary intersection d_b is computed in this step. This delay is incurred by bicyclists who travel through the intersection in the same lane as (or in a bicycle lane that is parallel to) the lanes used by segment through vehicles.

If the boundary intersection is two-way STOP controlled (where the subject approach is uncontrolled), the delay is equal to 0.0 s/bicycle. If the boundary intersection is signalized, the delay is computed by using the motorized vehicle methodology described in Chapter 19, Signalized Intersections.

Step 3: Determine Bicycle Travel Speed

Bicycle travel speed represents an aggregate measure of speed along the segment. It combines the delay incurred at the downstream boundary intersection and the time required to ride the length of the segment. Thus, it is typically slower than the average bicycle running speed. The average bicycle travel speed is computed by using Equation 18-40:

$$S_{Tb,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rb} + d_b)}$$

Equation 18-40

where

$S_{Tb,seg}$ = travel speed of through bicycles along the segment (mi/h),

L = segment length (ft),

t_{Rb} = segment running time of through bicycles = (3,600 L)/(5,280 S_b) (s),

S_b = bicycle running speed (mi/h), and

d_b = bicycle control delay (s/bicycle).

In general, a travel speed of 10.0 mi/h or more is considered desirable and a speed of 5.0 mi/h or less is considered undesirable.

Step 4: Determine Bicycle LOS Score for Intersection

The bicycle LOS score for the boundary intersection $I_{b,int}$ is determined in this step. If the boundary intersection is signalized, the bicycle methodology described in Chapter 19 is used for this determination. If the boundary intersection is two-way STOP controlled, the score is equal to 0.0.

Step 5: Determine Bicycle LOS Score for Link

The bicycle LOS score for the segment $I_{b,\text{link}}$ is calculated by using Equation 18-41:

Equation 18-41

$$I_{b,\text{link}} = 0.760 + F_w + F_v + F_s + F_p$$

with

Equation 18-42

$$F_w = -0.005\, W_e^2$$

Equation 18-43

$$F_v = 0.507 \ln\left(\frac{v_{ma}}{4\, N_{th}}\right)$$

Equation 18-44

$$F_S = 0.199\,[1.1199 \ln(S_{Ra} - 20) + 0.8103](1 + 0.1038\, P_{HVa})^2$$

Equation 18-45

$$F_p = \frac{7.066}{P_c^2}$$

where

$I_{b,\text{link}}$ = bicycle LOS score for link,

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_p = pavement condition adjustment factor,

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

W_e = effective width of outside through lane (see Exhibit 18-24) (ft),

v_{ma} = adjusted midsegment demand flow rate (see Exhibit 18-24) (veh/h),

N_{th} = number of through lanes on the segment in the subject direction of travel (ln),

S_{Ra} = adjusted motorized vehicle running speed (see Exhibit 18-24) (mi/h),

P_{HVa} = adjusted percent heavy vehicles in midsegment demand flow rate (see Exhibit 18-24) (%), and

P_c = pavement condition rating (see Exhibit 18-22).

The value used for several of the variables in Equation 18-42 to Equation 18-45 is dependent on various conditions. These conditions are identified in Column 1 of Exhibit 18-24. If the condition is satisfied, the equation in Column 2 is used to compute the variable value. If it is not satisfied, the equation in Column 3 is used. The equations in the first three rows are considered in sequence to determine the effective width of the outside through lane W_e.

The motorized vehicle running speed is computed by using the motorized vehicle methodology described in a previous subsection.

Exhibit 18-24 Variables for Bicycle LOS Score for Link

Condition	Variable When Condition Is Satisfied	Variable When Condition Is Not Satisfied
$p_{pk} = 0.0$	$W_t = W_{ol} + W_{bl} + W_{os}^* + W_{pk}$	$W_t = W_{ol} + W_{bl} + W_{os}^*$
$v_m > 160$ veh/h or street is divided	$W_v = W_t$	$W_v = W_t(2 - 0.005 v_m)$
$W_l < 4.0$ ft	$W_e = W_v - 10p_{pk} \geq 0.0$	$W_e = W_v + W_l - 20p_{pk} \geq 0.0$
$v_m(1 - 0.01\ P_{HV}) < 200$ veh/h and $P_{HV} > 50\%$	$P_{HVa} = 50\%$	$P_{HVa} = P_{HV}$
$S_R < 21$ mi/h	$S_{Ra} = 21$ mi/h	$S_{Ra} = S_R$
$v_m > 4\ N_{th}$	$v_{ma} = v_m$	$v_{ma} = 4N_{th}$

Notes: W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);
W_{ol} = width of outside through lane (ft);
W_{os}^* = adjusted width of paved outside shoulder; if curb is present $W_{os}^* = W_{os} - 1.5 \geq 0.0$, otherwise $W_{os}^* = W_{os}$ (ft);
W_{os} = width of paved outside shoulder (ft);
W_{bl} = width of bicycle lane = 0.0 if bicycle lane not provided (ft);
W_{pk} = width of striped parking lane (ft);
W_l = total width of shoulder, bicycle lane, and parking lane $= W_{bl} + W_{os}^* + W_{pk}$ (ft);
W_v = effective total width of outside through lane, bicycle lane, and shoulder as a function of traffic volume (ft);
p_{pk} = proportion of on-street parking occupied (decimal);
v_m = midsegment demand flow rate (veh/h);
P_{HV} = percent heavy vehicles in the midsegment demand flow rate; and
S_R = motorized vehicle running speed (mi/h).

Step 6: Determine Link LOS

The bicycle LOS for the link is determined by using the bicycle LOS score from Step 5. This score is compared with the link-based bicycle LOS thresholds in Exhibit 18-3 to determine the LOS for the specified direction of travel along the subject link.

Step 7: Determine Bicycle LOS Score for Segment

The bicycle LOS score for the segment is computed by using Equation 18-46:

Equation 18-46

$$I_{b,seg} = 0.75\left[\frac{\left(F_c + I_{b,\text{link}} + 1\right)^3 t_{R,b} + \left(I_{b,int} + 1\right)^3 d_b}{t_{Rb} + d_b}\right]^{\frac{1}{3}} + 0.125$$

with

Equation 18-47

$$F_c = 0.035\left(\frac{5{,}280\ N_{ap,s}}{L} - 20\right)$$

where

$I_{b,seg}$ = bicycle LOS score for segment;

$I_{b,\text{link}}$ = bicycle LOS score for link;

F_c = unsignalized conflicts factor;

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

$N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points).

The count of access point approaches used in Equation 18-46 includes both public street approaches and driveways on the right side of the segment in the subject direction of travel.

Step 8: Determine Segment LOS

The bicycle LOS for the segment is determined by using the segment bicycle LOS score from Step 7. This score is compared with the segment-based bicycle LOS thresholds in Exhibit 18-3 to determine the LOS for the specified direction of travel along the subject segment.

6. TRANSIT METHODOLOGY

This section describes the methodology for evaluating the capacity and quality of service provided to transit passengers on urban street segments.

SCOPE OF THE METHODOLOGY

The overall scope of the four methodologies was provided in Section 2. In addition, the transit methodology is limited to the evaluation of public transit vehicles operating in mixed or exclusive traffic lanes and stopping along the street. It is not designed to evaluate the performance of other travel means (e.g., grade-separated rail transit).

Spatial Limits

Travel Directions to Be Evaluated

Urban street segment performance from a transit passenger perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.*

Route-Based Evaluation

The methodology is used to evaluate a single transit route on the segment. If multiple routes exist on the segment, each route is evaluated by using a separate application of the methodology.

Performance Measures

Performance measures applicable to the transit travel mode include transit vehicle travel speed, transit wait–ride score, and transit LOS score. The LOS score is an indication of the typical transit rider's perception of the overall travel experience.

LOS is also considered a performance measure. It is useful for describing segment performance to elected officials, policy makers, administrators, or the public. LOS is based on the transit LOS score.

Limitations of the Methodology

In general, the methodology can be used to evaluate the performance of most urban street segments. However, it does not address all conditions or types of control. The inability to replicate the influence of a condition or control type in the methodology is a limitation.

This subsection identifies the known limitations of the transit methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific street segment, the analyst should consider using alternative methods or tools for the evaluation.

The transit methodology does not account for the effect of the following conditions on the quality of service provided to transit passengers:

- Presence of railroad crossings, and
- Transit vehicles on grade-separated or non-public-street rights-of-way.

Procedures for estimating transit vehicle performance on grade-separated or non-public-street rights-of-way, along with procedures for estimating origin–destination service quality, are provided in the *Transit Capacity and Quality of Service Manual* (*7*).

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the transit methodology. The required data are listed in Exhibit 18-25. They must be separately specified for each direction of travel on the segment and for each boundary intersection. The exhibit also lists default values that can be used if local data are not available (*2, 3*).

Exhibit 18-25
Required Input Data, Potential Data Sources, and Default Values for Transit Analysis

Required Data and Units	Potential Source(s)	Suggested Default Value
	Traffic Characteristics	
Dwell time (s)	Field data, AVL data	60 s (downtown stop, transit center, major on-line transfer point, major park-and-ride) 30 s (major outlying stop) 15 s (typical outlying stop)
Excess wait time (min)	Field data, AVL data	See discussion in text
Passenger trip length (mi)	National Transit Database	3.7 mi
Transit frequency (veh/h)	Transit schedules	Must be provided
Passenger load factor (p/seat)	Field data, APC data	0.80 p/seat
	Geometric Data	
Segment length[a] (ft)	Field data, aerial photo	Must be provided
	Other Data	
CBD of 5-million-plus metro area (yes/no)	Census data	Must be provided
Traffic signal effective green-to-cycle-length ratio (decimal)	Field data or HCM method output	Must be provided (if present)
Traffic signal cycle length (s)	Field data or HCM method output	Must be provided (if present)
Transit stop location (nearside/other)	Field data, aerial photo	Must be provided
Transit stop position (on-line/off-line)	Field data, aerial photo	Must be provided
Proportion of transit stops with shelters (decimal)	Field data, transit facility inventory	Must be provided
Proportion of transit stops with benches (decimal)	Field data, transit facility inventory	Must be provided
	Performance Measures	
Motorized vehicle running time[a] (s)	HCM method output	Must be provided
Pedestrian LOS score for link (decimal)	HCM method output	Must be provided
Reentry delay (s/veh)	HCM method output	Must be provided
Roundabout volume-to-capacity ratio (decimal)	HCM method output	Must be provided (if present)

Notes: AVL = automatic vehicle location, APC = automatic passenger counter, CBD = central business district.
[a] Also used or calculated by the motorized vehicle methodology.

The data elements listed in Exhibit 18-25 do not include variables that are considered to represent calibration factors. A calibration factor typically has a relatively narrow range of reasonable values or has 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.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 18-25. These data describe the transit traffic streams traveling along the segment during the analysis period. If there are multiple transit routes on the segment, the transit-related variables are needed for each route.

Dwell Time

Dwell time is the time that the transit vehicle is stopped at the curb to serve passenger movements, including the time required to open and close the doors. It does not include time spent stopped after passenger movements have ceased (e.g., waiting for a traffic signal or waiting for a gap in traffic to reenter the travel lane). Dwell times are typically in the range of 10 to 60 s, depending on boarding and alighting demand. Procedures for measuring and estimating dwell time are provided in the *Transit Capacity and Quality of Service Manual* (*7*).

Excess Wait Time

Transit reliability is measured by *excess wait time,* the average number of minutes passengers must wait at a stop past the scheduled departure time. It is measured in the field as the sum of the differences between the scheduled and actual departure times at the preceding time point, divided by the number of transit vehicle arrivals. Early departures from the preceding time point are treated as the transit vehicle being one headway late, since a passenger arriving at the stop by the scheduled departure time would have to wait one headway for the next transit vehicle. If time point–specific excess wait time information is not available but on-time performance (e.g., percentage of departures from a time point 0 to 5 min late) data are available for a route, the methodology provides a procedure for estimating excess wait time from on-time performance.

The scheduled departure time from a stop and the scheduled travel time for a trip set the baseline for a passenger's expectations for how long a trip should take. If the transit vehicle departs late—or worse, departs before the scheduled time (i.e., before all the passengers planning to take that vehicle have arrived at the stop)—the trip will likely take longer than planned, which negatively affects a passenger's perceptions of the quality of service.

Passenger Trip Length

For most purposes, the average trip length can be determined from National Transit Database data for the transit agency (*8*) by dividing total passenger miles by total unlinked trips. However, if an analyst has reason to believe that average trip length on a route is substantially different from the system average, a route-specific value can be determined from automatic passenger counter data or National Transit Database count sheets for the route by dividing total passenger miles by the total number of boarding passengers.

The impact of a late transit vehicle departure on the overall passenger speed for a trip (as measured by using scheduled departure time to actual arrival time) depends on the length of the passenger's trip. For example, a departure 5 min late has more of a speed impact on a 1-mi-long trip than on a 10-mi-long trip.

Average passenger trip length is used to determine the impact of late departures on overall trip speed.

Transit Frequency

Transit frequency is defined as the count of scheduled fixed-route transit vehicles that stop on or near the segment during the analysis period. It is expressed in units of transit vehicles per hour.

Scheduled transit vehicles can be considered "local" or "nonlocal." Local transit vehicles make regular stops along the street (typically every 0.25 mi or less), although they do not necessarily stop within the analysis segment when segment lengths are short or when stops alternate between the near and far sides of boundary intersections. They are always counted, regardless of whether they stop within the subject segment. Nonlocal transit vehicles operate on routes with longer stop spacing than local routes (e.g., limited-stop, bus rapid transit, or express routes). They are only counted when they stop within the subject segment.

Passenger Load Factor

The load factor is the number of passengers occupying the transit vehicle divided by the number of seats on the vehicle. If the number of passengers equals the number of seats, the load factor equals 1.0. This factor should be measured in the field or obtained from the agency serving the transit route. It is an average value for all of the scheduled fixed-route transit vehicles that travel along the segment during the analysis period.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 18-25. These data describe the geometric elements that influence the service provided to transit passengers. All input data should be representative of the segment for its entire length. An average value should be used for each element that varies along the segment.

Segment length is the only variable in this category. It is defined in a similarly titled section for the motorized vehicle methodology.

Other Data

This subsection describes the data listed in Exhibit 18-25 that are categorized as "other data."

Area Type

Area type describes the environment in which the subject segment is located. This data element is used in the transit methodology to set a baseline for passenger expectations of typical transit travel speeds. For this application, it is sufficient to indicate whether the area type is a "central business district of a metropolitan area with over 5 million persons" or "other."

Effective Green-to-Cycle-Length Ratio and Cycle Length

The cycle length and the effective green-to-cycle-length ratio for the through movement are used in the transit methodology when the boundary intersection is a traffic signal. If the signal is actuated, the motorized vehicle methodology in Chapter 19 can be used to estimate the average green-to-cycle-length ratio and cycle length.

Transit Stop Location

This input describes whether a transit stop is located on the near side of a boundary intersection or elsewhere. A portion of the time required to serve a near-side transit stop at a boundary intersection may overlap with the control delay incurred at the intersection.

Transit Stop Position

Transit stops can be either *on-line*, where the bus stops entirely or mostly in the travel lane and does not have to yield to other vehicles on exiting the stop, or *off-line,* where the bus pulls out of the travel lane to serve the stop and may have to yield to other vehicles on exiting.

Proportion of Stops with Shelters and with Benches

These two input data elements describe the passenger amenities provided at a transit stop. A sheltered stop provides a structure with a roof and three enclosing sides that protect occupants from wind, rain, and sun. A shelter with a bench is counted twice, once as a shelter and a second time as a bench.

Performance Measures

This subsection describes the data listed in Exhibit 18-25 that are categorized as "performance measures."

Motorized Vehicle Running Time

The motorized vehicle running time for the segment is obtained from the motorized vehicle methodology that is described in Section 3.

Pedestrian LOS Score for Link

The pedestrian LOS score for the link is obtained from the pedestrian methodology that is described in Section 4.

Reentry Delay

The final component of transit vehicle stop delay is the reentry delay, the time (in seconds) a transit vehicle spends waiting for a gap to reenter the adjacent traffic stream. Reentry delay is estimated as follows (*7*):

- Reentry delay is zero at on-line stops.
- At off-line stops away from the influence of a signalized intersection queue, reentry delay is estimated from the procedures of Chapter 20, Two-Way STOP-Controlled Intersections, as if the bus were making a right turn onto the link, but a critical headway of 7 s is used to account for the slower acceleration of buses.

- At an off-line bus stop located within the influence of a signalized intersection queue, reentry delay is estimated from the queue service time, g_s, by using the motorized vehicle methodology in Chapter 19, Signalized Intersections.

Reentry delay can be reduced by the presence of yield-to-bus laws or placards (and motorist compliance with them), the existence of an acceleration lane or queue jump departing a stop, or a higher-than-normal degree of bus driver aggressiveness in forcing buses back into the traffic stream. Analyst judgment and local data can be used to make appropriate adjustments to reentry delay in these cases.

Volume-to-Capacity Ratio (If Roundabout)

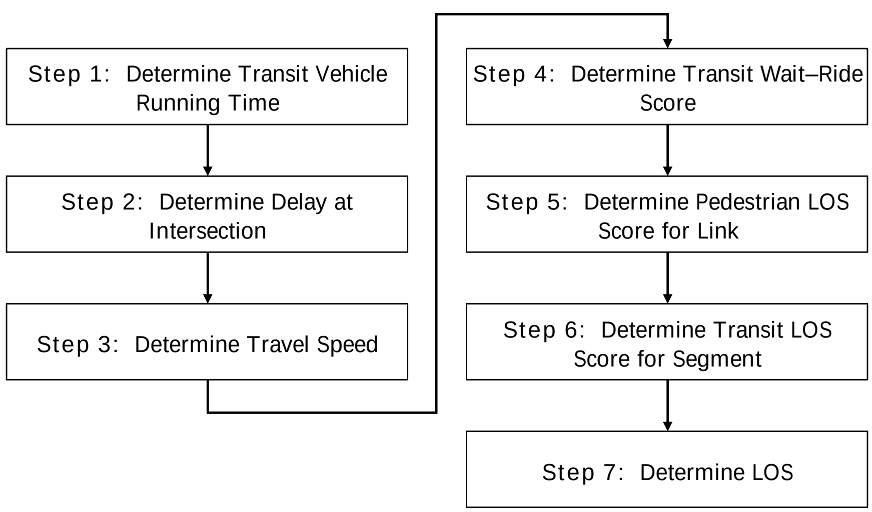
If the boundary intersection is a roundabout and it has a near-side transit stop, the volume-to-capacity ratio for the rightmost lane of the segment approach to the roundabout is needed. It is obtained from the Chapter 22 methodology.

OVERVIEW OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of an urban street segment in terms of its service to transit passengers.

The transit methodology is applied through a series of seven steps that culminate in the determination of segment LOS. These steps are illustrated in Exhibit 18-26. Performance measures that are estimated include transit travel speed along the street, transit wait–ride score, and a LOS score reflective of all transit service stopping within or near the segment.

Exhibit 18-26
Transit Methodology for Urban Street Segments



COMPUTATIONAL STEPS

Step 1: Determine Transit Vehicle Running Time

There are two principal components of the transit vehicle's segment running time. One is the time required to travel the segment without stopping. (To allow direct comparison with automobile segment speeds, transit vehicles are treated as if they travel the entire segment, even if they join midlink.) The second is the delay incurred at the transit stops that are provided on the link. The following subparts to this step describe procedures that are used to calculate these components. They culminate with a subsection that describes the calculation of transit vehicle segment running time.

A. Compute Segment Running Speed

Transit vehicle segment running speed is the speed reached by the vehicle when it is not influenced by the proximity of a transit stop or traffic control device. This speed can be computed with Equation 18-48, which is derived from tables given in a Transit Cooperative Research Program report (*9*).

$$S_{Rt} = \min\left(S_R, \frac{61}{1 + e^{-1.00 + (1{,}185\, N_{ts}/\, L)}}\right)$$

Equation 18-48

where

S_{Rt} = transit vehicle running speed (mi/h),

L = segment length (ft),

N_{ts} = number of transit stops on the segment for the subject route (stops),

S_R = motorized vehicle running speed = (3,600 L)/(5,280 t_R) (mi/h), and

t_R = segment running time (s).

The segment running time is computed by using Equation 18-7 in Step 2 of the motorized vehicle methodology.

B. Compute Delay due to a Stop

The delay due to a transit vehicle stop for passenger pickup includes the following components:

- Acceleration–deceleration delay,
- Delay due to serving passengers, and
- Reentry delay.

This procedure is applied once for each stop on the segment. The delay due to each stop is added (in a subsequent step) to compute the total delay due to all stops on the segment.

Acceleration–Deceleration Delay

Acceleration–deceleration delay is the additional time required to decelerate to stop and then accelerate back to the transit vehicle running speed S_{Rt}. It is computed with Equation 18-49 and Equation 18-50.

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{S_{Rt}}{2}\right)\left(\frac{1}{r_{at}} + \frac{1}{r_{dt}}\right) f_{ad}$$

Equation 18-49

with

$$f_{ad} = \begin{cases} 1.00 & \text{(stops not on the near side of a boundary intersection)} \\ 0.00 & \text{(near-side stops at all-way and major-street two-way STOP-controlled intersections)} \\ 1 - x & \text{(near-side stops at roundabouts)} \\ g/C & \text{(near-side stops at traffic signals)} \end{cases}$$

Equation 18-50

where

d_{ad} = transit vehicle acceleration–deceleration delay due to a transit stop (s),

r_{at} = transit vehicle acceleration rate = 3.3 (ft/s^2),

r_{dt} = transit vehicle deceleration rate = 4.0 (ft/s^2),

f_{ad} = proportion of transit vehicle stop acceleration–deceleration delay not due to traffic control,

x = volume-to-capacity ratio of the link's rightmost lane on a roundabout approach,

g = effective green time (s), and

C = cycle length (s).

Acceleration–deceleration delay represents travel time that is in excess of that required to traverse the equivalent distance at the running speed. It is incurred when the transit vehicle stops solely because of a transit stop. When a transit stop is located on the near side of a boundary intersection, a transit vehicle might need to stop anyway due to the traffic control. In this situation, acceleration–deceleration delay is already included in the through delay estimate (addressed in a subsequent step) and should not be included in d_{ad}. Equation 18-50 is used to determine the proportion of d_{ad} incurred solely because of a transit stop.

If representative acceleration and deceleration rates are known, they should be used in Equation 18-49. If these rates are unknown, an acceleration rate of 3.3 ft/s^2 and a deceleration rate of 4.0 ft/s^2 can be used (*7*).

Delay due to Serving Passengers

The delay due to serving passengers is based on the average dwell time, which is an input to this procedure. At signalized intersections, a portion of the dwell time may overlap time the transit vehicle would have spent stopped anyway due to the traffic control. Equation 18-51 is used to compute the delay due to serving passengers.

Equation 18-51

$$d_{ps} = t_d\, f_{dt}$$

where

d_{ps} = transit vehicle delay due to serving passengers (s),

t_d = average dwell time (s), and

f_{dt} = proportion of dwell time occurring during effective green (= g/C at near-side stops at signalized intersections and 1.00 otherwise, where g and C are as previously defined).

Reentry Delay

The final component of transit vehicle stop delay is the reentry delay d_{re}, which is an input to this procedure. Guidance for estimating reentry delay is provided in the Required Data and Sources section.

Delay due to a Stop

Delay due to a transit stop is the sum of acceleration–deceleration delay, passenger service time delay, and reentry delay. It is computed with Equation 18-52.

$$d_{ts} = d_{ad} + d_{ps} + d_{re}$$

Equation 18-52

where d_{ts} is the delay due to a transit vehicle stop (s), d_{re} is the reentry delay (s), and all other variables are as previously defined.

C. Compute Segment Running Time

Equation 18-53 is used to compute transit vehicle running time, which is based on segment running speed and delay due to stops on the segment.

$$t_{Rt} = \frac{3{,}600\ L}{5{,}280\ S_{Rt}} + \sum_{i=1}^{N_{ts}} d_{ts,i}$$

Equation 18-53

where t_{Rt} is the segment transit vehicle running time (s), $d_{ts,i}$ is the delay due to a transit vehicle stop for passenger pickup at stop *i* within the segment (s), and all other variables are as previously defined.

If there are no stops on the segment, the second term of Equation 18-53 equals zero.

Step 2: Determine Delay at Intersection

The through delay d_t incurred at the boundary intersection by the transit vehicle is determined in this step. This delay is equal to the control delay incurred by through vehicles that exit the segment at the downstream boundary intersection. Guidance for determining this delay is provided in Step 5 of the motorized vehicle methodology.

Alternatively, Equation 18-54 can be used to estimate the through delay due to a traffic signal (*9*). This estimate is suitable for a planning-level analysis.

$$d_t = t_l\ 60 \left(\frac{L}{5{,}280}\right)$$

Equation 18-54

where

d_t = through delay (s/veh),

t_l = transit vehicle running time loss (min/mi), and

L = segment length (ft).

The running time loss t_l used in Equation 18-54 is obtained from Exhibit 18-27.

Exhibit 18-27
Transit Vehicle Running Time Loss

Area Type	Transit Lane Allocation	Traffic Condition	Running Time Loss by Signal Condition (min/mi): Typical	Signals Set for Transit	Signals More Frequent Than Transit Stops
Central business district	Exclusive	No right turns	1.2	0.6	1.5–2.0
		With right-turn delay	2.0	1.4	2.5–3.0
		Blocked by traffic	2.5–3.0	Not available	3.0–3.5
	Mixed traffic	Any	3.0	Not available	3.5–4.0
Other	Exclusive	Any	0.7 (0.5–1.0)	Not available	Not available
	Mixed traffic	Any	1.0 (0.7–1.5)	Not available	Not available

Source: St. Jacques and Levinson (*9*).

Step 3: Determine Travel Speed

Transit travel speed is an aggregate measure of speed along the street. It combines the delay incurred at the downstream intersection with the segment running time. Thus, it is typically slower than the running speed. The transit travel speed is computed by using Equation 18-55.

Equation 18-55

$$S_{Tt,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rt} + d_t)}$$

where $S_{Tt,seg}$ is the travel speed of transit vehicles along the segment (mi/h), t_{Rt} is the segment running time of transit vehicles (s), and all other variables are as previously defined.

Step 4: Determine Transit Wait–Ride Score

The transit wait–ride score is a performance measure that combines perceived time spent waiting for the transit vehicle and perceived travel time rate. If transit service is not provided for the subject direction of travel, this score equals 0.0 and the analysis continues with Step 5.

The procedure for calculating the wait–ride score is described in this step. It consists of the separate calculation of the headway factor and the perceived travel time factor. The following subsections describe these two calculations, which culminate in the calculation of the wait–ride score.

A. Compute Headway Factor

The headway factor is the ratio of the estimated patronage at the prevailing average transit headway to the estimated patronage at a base headway of 60 min. The patronage values for the two headways (i.e., the input headway and the base headway of 60 min) are computed from an assumed set of patronage elasticities that relate the percentage change in ridership to the percentage change in headway. The headway factor is computed by using Equation 18-56.

Equation 18-56

$$F_h = 4.00\ e^{-1.434/(v_s + 0.001)}$$

where

F_h = headway factor, and

v_s = transit frequency for the segment (veh/h).

The transit frequency v_s is an input to this procedure. Guidance for estimating this input is provided in the Required Data and Sources section.

B. Compute Perceived Travel Time Factor

Segment performance, as measured by the wait–ride score, is influenced by the travel time rate provided to transit passengers. The perceptibility of this rate is further influenced by the extent to which the transit vehicle is late, crowded, or both and whether the stop provides passenger amenities. In general, travel at a high rate is preferred, but travel at a lower rate may be nearly as acceptable if the transit vehicle is not late, the bus is lightly loaded, and a shelter (with a bench) is provided at the transit stop.

The perceived travel time factor is based on the perceived travel time rate and the expected ridership elasticity with respect to changes in the perceived travel time rate. This factor is computed with Equation 18-57.

$$F_{tt} = \frac{(e-1)\,T_{btt} - (e+1)\,T_{ptt}}{(e-1)\,T_{ptt} - (e+1)\,T_{btt}}$$

Equation 18-57

with

$$T_{ptt} = \left(a_1 \frac{60}{S_{Tt,seg}}\right) + (2\,T_{ex}) - T_{at}$$

Equation 18-58

$$a_1 = \begin{cases} 1.00 & F_l \le 0.80 \\ 1 + \dfrac{4\,(F_l - 0.80)}{4.2} & 0.80 \le F_l \le 1.00 \\ 1 + \dfrac{4\,(F_l - 0.80) + (F_l - 1.00)[6.5 + 5\,(F_l - 1.00)]}{4.2\,F_l} & F_l > 1.00 \end{cases}$$

Equation 18-59

$$T_{at} = \frac{1.3\,p_{sh} + 0.2\,p_{be}}{L_{pt}}$$

Equation 18-60

where

F_{tt} = perceived travel time factor;

e = ridership elasticity with respect to changes in the travel time rate = –0.40;

T_{btt} = base travel time rate = 6.0 for the central business district of a metropolitan area with 5 million persons or more, otherwise = 4.0 (min/mi);

T_{ptt} = perceived travel time rate (min/mi);

T_{ex} = excess wait time rate due to late arrivals (min/mi) = t_{ex}/L_{pt};

t_{ex} = excess wait time due to late arrivals (min);

T_{at} = amenity time rate (min/mi);

a_1 = passenger load weighting factor;

$S_{Tt,seg}$ = travel speed of transit vehicles along the segment (mi/h);

F_l = average passenger load factor (passengers/seat);

L_{pt} = average passenger trip length = 3.7 typically (mi);

p_{sh} = proportion of stops on segment with shelters (decimal); and

p_{be} = proportion of stops on segment with benches (decimal).

The perceived travel time rate is estimated according to three components, as shown in Equation 18-58. The first component reflects the average travel speed of the transit service, adjusted for the degree of passenger loading. The second component reflects the average excess wait time for the transit vehicle (i.e., the amount of time spent waiting for a late arrival beyond the scheduled arrival time). The third component reflects the ability of passengers to tolerate longer travel time rates when amenities are provided at the transit stops.

The first term in Equation 18-58 includes a factor that adjusts the transit vehicle travel time rate by using a passenger load weighting factor. This factor accounts for the decrease in passenger comfort when transit vehicles are crowded. Values of this factor range from 1.00 when the passenger load factor is less than 0.80 passengers/seat to 2.32 when the load factor is 1.6 passengers/seat.

The second term in Equation 18-58 represents the perceived excess wait time rate. It is based on the excess wait time t_{ex} associated with late transit arrivals. The multiplier of 2 in Equation 18-58 is used to amplify the excess wait time rate because passengers perceive excess waiting time to be more onerous than actual travel time.

The excess wait time t_{ex} reflects transit vehicle reliability. It is an input to this procedure. If excess wait time data are not available for a stop but on-time performance data are available for routes using the stop, Equation 18-61 may be used to estimate the average excess wait time.

Equation 18-61

$$t_{ex} = [t_{late}(1 - p_{ot})]^2$$

where

t_{ex} = excess wait time due to late arrivals (min),

t_{late} = threshold late time = 5.0 typical (min), and

p_{ot} = proportion of transit vehicles arriving within the threshold late time (default = 0.75) (decimal).

The third term in Equation 18-58 represents the amenity time rate reduction. This rate is computed in Equation 18-60 as the equivalent time value of various transit stop improvements divided by the average passenger trip length. If multiple transit stops exist on the segment, an average amenity time rate should be used for the segment, based on the average value for all stops in the segment.

The average passenger trip length is used to convert time values for excess wait time and amenities into distance-weighted travel time rates that adjust the perceived in-vehicle travel time rate. The shorter the trip, the greater the influence that late transit vehicles and stop amenities have on the overall perceived speed of the trip.

The average passenger trip length should be representative of transit routes using the subject segment. A value of 3.7 mi is considered to be nationally representative. More accurate local values can be obtained from the National Transit Database (*8*). Specifically, this database provides annual passenger miles and annual unlinked trips in the profile of most transit agencies. The average passenger trip length is computed as the annual passenger miles divided by the annual unlinked trips.

C. Compute Wait–Ride Score

The wait–ride score is computed with Equation 18-62. A larger score corresponds to better performance.

$$s_{w\text{-}r} = F_h \, F_{tt}$$

Equation 18-62

where

$s_{w\text{-}r}$ = transit wait–ride score,

F_h = headway factor, and

F_{tt} = perceived travel time factor.

Step 5: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link $I_{p,\text{link}}$ is computed by using the pedestrian methodology, as described in Section 4.

Step 6: Determine Transit LOS Score for Segment

The transit LOS score for the segment is computed by using Equation 18-63.

$$I_{t,seg} = 6.0 - 1.50\, s_{w\text{-}r} + 0.15\, I_{p,\text{link}}$$

Equation 18-63

where $I_{t,seg}$ is the transit LOS score for the segment and all other variables are as defined previously.

Step 7: Determine LOS

The transit LOS is determined by using the transit LOS score from Step 6. This performance measure is compared with the thresholds in Exhibit 18-3 to determine the LOS for the specified direction of travel along the subject street segment.

7. APPLICATIONS

EXAMPLE PROBLEMS

Chapter 30, Urban Street Segments: Supplemental, describes the application of each of the four methodologies through the use of example problems. There is one example problem associated with each methodology. The examples illustrate the operational analysis type.

GENERALIZED DAILY SERVICE VOLUMES

Generalized daily service volume tables provide a means of quickly assessing one or more urban street facilities to determine which facilities need to be more carefully evaluated (with operational analysis) to ameliorate existing or pending problems. Their application in practice is typically at the facility level rather than at the segment level. For this reason, service volume tables are provided in Chapter 16, Urban Street Facilities.

ANALYSIS TYPE

The four methodologies described in this chapter can each be used in three types of analysis. The analysis types are described as operational, design, and planning and preliminary engineering. The selected analysis type applies to the methodology described in this chapter and to all supporting methodologies. The characteristics of each analysis type are described in the subsequent paragraphs.

Operational Analysis

The objective of an operational analysis is to determine the LOS for current or near-term conditions when details of traffic volumes, geometry, and traffic control conditions are known. All the methodology steps are implemented and all calculation procedures are applied for the purpose of computing a wide range of performance measures. The operational analysis type will provide the most reliable results because it uses no (or minimal) default values.

Design Analysis

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

The nature of the design analysis type depends on whether the boundary intersections are unsignalized or signalized. When the segment has unsignalized boundary intersections, the analyst specifies traffic conditions and target levels for a set of performance measures. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

When the segment has signalized boundary intersections, the design analysis type has two variations. Both variations require the specification of traffic conditions and target levels for a set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

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

Planning and Preliminary Engineering Analysis

The objective of a planning and preliminary engineering analysis can be (*a*) to determine the LOS for either a proposed segment or an existing segment in a future year or (*b*) to size the overall geometrics of a proposed segment.

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

The requirement for a complete description of the signal timing plan can be a burden for some planning analyses involving signalized intersections. The intersection planning-level analysis application described in Chapter 31, Signalized Intersections: Supplemental, can be used to estimate a reasonable timing plan, in conjunction with the aforementioned default values.

For some planning and preliminary engineering analyses, the segment planning-level analysis application described in Chapter 30, Urban Street Segments: Supplemental, may provide a better balance between accuracy and analysis effort in the evaluation of vehicle LOS.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This subsection contains specific guidance for the application of alternative tools to the analysis of urban street segments. Additional information on this topic is provided in the Technical Reference Library in Volume 4. The focus of this subsection is the application of alternative tools to evaluate motorized vehicle operation.

Comparison of Motorized Vehicle Methodology and Alternative Tools

Motorized Vehicle Methodology

The motorized vehicle methodology models the driver–vehicle–road system with reasonable accuracy for most applications. It accounts for signal coordination, platoon dispersion, the origin–destination patterns of all segment traffic flows, driveway impacts on traffic flow, and the influence of volume on speed.

The motorized vehicle methodology offers several advantages over alternative analysis tools. One advantage is that it has an empirically calibrated procedure for estimating saturation flow rate. Alternative tools often require saturation flow rate as an input variable. A second is that it produces a direct estimate of capacity and volume-to-capacity ratio. These measures are not directly available from simulation tools. A third advantage is that it produces an expected value for each of several performance measures in a single application. Simulation tools require multiple runs and manual calculations to obtain an expected value for a given performance measure. A fourth is that its analytic procedures are described in the HCM so that analysts can understand the driver–vehicle–road interactions and the means by which they are modeled. Most proprietary alternative tools operate as a "black box," providing little detail describing the intermediate calculations.

Alternative Tools

Both deterministic tools and simulation tools are in common use as alternatives to the motorized vehicle methodology offered in this chapter. Deterministic tools are often used for the analysis of urban street segments. The main reasons for their popularity are found in the user interface, optimization options, and output presentation features. Some also offer additional performance measures such as fuel consumption, air quality, and operating cost.

Conceptual Differences

Alternative deterministic tools apply traffic models that are conceptually similar to those described in this chapter. While their computational details will usually produce different numerical results, there are few major conceptual differences that would preclude comparison of commonly defined performance measures.

Simulation tools, on the other hand, are based on entirely different modeling concepts. A general discussion of the conceptual differences is presented in Chapters 6 and 7. Some specific examples for signalized intersections, which also apply to urban street segments, are presented in Section 7 of Chapter 19.

One phenomenon that makes comparison difficult is the propagation of platoons along a segment. Deterministic tools, including the model presented in this chapter, apply equations that spread out a platoon as it progresses downstream. Simulation tools create platoon dispersion implicitly from a distribution of desired speeds among drivers. Both approaches will produce platoon dispersion, but the amount of dispersion will differ among tools.

Simulation tools may also exhibit platoon compression because of the effect of slower-moving vehicles that cause platoons to regenerate. For this and other reasons, comparability of platoon representation along a segment between these tools and the motorized vehicle methodology is difficult to achieve.

Alternative Tool Application Guidance

Development of HCM-Compatible Performance Measures

Alternative tools generally define travel speed in the same way that it is defined in this chapter. However, these tools may not compute delay and running speed by using the procedures presented in this chapter. Therefore, care must be taken in comparing speed and delay estimates from this chapter with those from other tools. Issues related to the comparison of speed (or delay) among different tools are discussed in more detail in Chapter 7. In general, the travel speed from an alternative tool should not be used for LOS assessment unless the tool is confirmed to apply the definitions and procedures described in this chapter.

Adjustment of Parameters

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

Adjustment of the alternative tool parameters that affect the travel time along the segment might also be necessary to produce comparable results. The motorized vehicle methodology is based on a free-flow speed that is computed as a function of demand flow rate, median type, access point density, parking presence, and speed limit. Most alternative tools typically require a user-specified free-flow speed, which could be obtained from the motorized vehicle methodology to maintain comparability. Adjustment of the platoon modeling parameters may be more difficult. Thus, if comparability is desired in representing the platoon effect, it is preferable to adjust the free-flow speed specified for simulation so that the actual travel speeds are similar to those obtained from the motorized vehicle methodology.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 29, Urban Street Facilities: Supplemental, includes a set of examples illustrating the use of alternative tools to address the stated limitations of this chapter and Chapter 16, Urban Street Facilities. Specifically, the examples illustrate (*a*) the application of deterministic tools to optimize signal timing, (*b*) the effect of platooned arrivals at a roundabout, (*c*) the effect of midsegment parking maneuvers on facility operation, and (*d*) the use of simulated vehicle trajectories to evaluate the proportion of time that the back of the queue on the minor-street approach to a two-way STOP-controlled intersection exceeds a specified distance from the stop line.

Chapter 31, Signalized Intersections: Supplemental, includes example problems that address left-turn storage bay overflow, right-turn-on-red operation, short through lanes, and closely spaced intersections.

8. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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Enhancing Pedestrian Volume Estimation and Developing HCM Pedestrian Methodologies for Safe and Sustainable Communities

Chapter 19: Signalized Intersections

Version 6.1 Final Draft

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This chapter contains revisions to the pedestrian delay methodologies for signalized intersections presented in Section 5, along with small changes in Section 1 (Introduction) and to descriptions of default pedestrian speeds in Section 3 (Motorized Vehicle Methodology) for consistency with Section 5. These revisions were developed through NCHRP Project 17-87.

CHAPTER 19
SIGNALIZED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

This chapter describes separate methodologies for evaluating the operation of each of the following intersection travel modes: motorized vehicle, pedestrian, and bicycle. Each methodology is used to evaluate the quality of service provided to road users traveling through a signalized intersection. A detailed description of each travel mode is provided in Chapter 2, Applications.

The methodologies are much more than just a means of evaluating quality of service. They include an array of performance measures that fully describe intersection operation. These measures serve as clues for identifying operational issues. They also provide insight into the development of effective improvement strategies. The analyst is encouraged to consider the full range of performance measures associated with each methodology.

This chapter also describes methodologies for evaluating intersection performance from the perspective of motorists, pedestrians, and bicyclists. These methodologies are referred to as the motorized vehicle methodology, pedestrian methodology, and bicycle methodology. Collectively, they can be used to evaluate the intersection operation from a multimodal perspective.

Each methodology in this chapter focuses on the evaluation of a signalized intersection. Chapter 18, Urban Street Segments, provides a methodology for quantifying the performance of an urban street segment. The methodology described in Chapter 16, Urban Street Facilities, can be used to combine the performance measures (for a specified travel mode) on successive segments into an overall measure of facility performance for that mode.

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 volume-to-capacity ratio, motorized vehicle control delay, pedestrian corner circulation area, pedestrian delay, pedestrian level-of-service (LOS) score, bicycle delay, and bicycle LOS score.

The motorized vehicle methodology has evolved and reflects the findings from a large body of research. It was originally based, in part, on the results of a National Cooperative Highway Research Program (NCHRP) study (*1*, *2*) that formalized the critical movement analysis procedure and the motorized vehicle delay estimation procedure. The critical movement analysis procedure was developed in the United States (*3*, *4*), Australia (*5*), Great Britain (*6*), and Sweden (*7*). The motorized vehicle delay estimation procedure was developed in Great Britain (*8*), Australia (*9*), and the United States (*10*). Updates to the original methodology were developed in a series of research projects (*11–24*).

The procedures for evaluating pedestrian and bicyclist perception of LOS are documented in an NCHRP report (*25*). The procedures for evaluating pedestrian delay, pedestrian circulation area, and bicyclist delay are documented in two Federal Highway Administration reports (*26*, *27*).

CHAPTER ORGANIZATION

Section 2 of this chapter presents concepts used to describe signalized intersection operation. It provides an overview of traffic signal-timing and phasing concepts. It also includes guidance for establishing the intersection analysis boundaries and the analysis period duration. It concludes with a discussion of the service measures and LOS thresholds used in the methodology.

Section 3 presents the core methodology for evaluating motorized vehicle service at a signalized intersection. The presentation describes the scope of the methodology and its required input data. It concludes with a description of the computational steps that are followed for each application of the methodology.

Section 4 describes extensions to the core motorized vehicle methodology, including the calculation of intersection volume-to-capacity ratio, uniform delay calculation, and initial queue delay calculation.

Section 5 presents the methodology for evaluating pedestrian service at a signalized intersection. The presentation includes a discussion of methodology scope, input data, and computational steps.

Section 6 presents the methodology for evaluating bicycle service at a signalized intersection. The presentation includes a discussion of methodology scope, input data, and computational steps.

Section 7 presents guidance on using the results of the intersection evaluation. The presentation includes example results from each methodology and a discussion of situations in which alternative evaluation tools may be appropriate.

RELATED HCM CONTENT

Other *Highway Capacity Manual* (HCM) content related to this chapter includes

- Chapter 18, Urban Street Segments, which describes concepts and methodologies for the evaluation of an urban street segment;
- Chapter 29, Urban Street Facilities: Supplemental, which provides details of the reliability methodology, a procedure for sustained spillback analysis, information about the use of alternative evaluation tools, and example problems demonstrating both the urban street facility methodologies and the reliability methodology;
- Chapter 30, Urban Street Segments: Supplemental, which describes procedures for predicting platoon flow, spillback, and delay due to turns from the major street; a planning-level analysis application; and example problems demonstrating the urban street segment methodologies;
- Chapter 31, Signalized Intersections: Supplemental, which describes procedures for predicting actuated phase duration, lane volume distribution, queue length, and saturation flow adjustment factors for pedestrian, bicycle, and work zone presence; a planning-level analysis application; and example problems demonstrating the signalized intersection methodologies;

- Case Study 1, U.S. 95 Corridor; Case Study 2, Route 146 Corridor; and Case Study 3, Krome Avenue, in the *HCM Applications Guide* in Volume 4, which demonstrate how this chapter's methods can be applied to the evaluation of actual signalized intersections; and
- Section L, Signalized Intersections, in Part 2 of the *Planning and Preliminary Engineering Applications Guide to the HCM*, found in Volume 4, which describes how to incorporate this chapter's methods and performance measures into a planning effort.

A procedure for determining intersection saturation flow rate when a work zone is present upstream (or downstream) of the intersection is provided in the final report for NCHRP Project 03-107, Work Zone Capacity Methods for the HCM. This report is in online Volume 4.

2. CONCEPTS

This section presents concepts used to describe signalized intersection operation. The first subsection describes basic signalized intersection concepts, including traffic signal control, the traffic movement numbering scheme, phase sequence and operational modes, intersection traffic flow characteristics, and phase duration components. The second identifies the types of analysis that can be conducted. The third includes guidance for establishing the intersection analysis boundaries and the analysis period duration. The fourth discusses the service measures and LOS thresholds used in the methodology, and the last subsection identifies the scope of the collective set of methodologies.

TRAFFIC SIGNAL CONCEPTS

Types of Traffic Signal Control

In general, two types of traffic signal controller are in use today. They are broadly categorized as pretimed or actuated according to the type of control they provide. These two types of control are described as follows:

- *Pretimed control* consists of a fixed sequence of phases that are displayed in repetitive order. The duration of each phase is fixed. However, the green interval duration can be changed by time of day or day of week to accommodate traffic variations. The combination of a fixed phase sequence and fixed duration produces a constant cycle length.
- *Actuated control* consists of a defined phase sequence in which the presentation of each phase depends on whether the phase is on recall or the associated traffic movement has submitted a call for service through a detector. The green interval duration is determined by the traffic demand information obtained from the detector, subject to preset minimum and maximum limits. The termination of an actuated phase requires a call for service from a conflicting traffic movement. An actuated phase may be skipped if no demand is detected.

Most modern controllers have solid-state components and use software to implement the control logic. This architecture is sufficiently flexible to provide either actuated control or pretimed control.

The operation of a pretimed controller can be described as coordinated or not coordinated. In contrast, the operation of an actuated controller can be described as fully actuated, semiactuated, or coordinated-actuated. These actuated control variations are described as follows:

- *Fully actuated control* implies that all phases are actuated and all intersection traffic movements are detected. The sequence and duration of each phase are determined by traffic demand. Hence, this type of control is *not* associated with a constant cycle length.
- *Semiactuated control* uses actuated phases to serve the minor movements at an intersection. Only these minor movements have detection. The phases associated with the major movements are operated as “nonactuated.” The controller is programmed to dwell with the nonactuated phases

displaying green for at least a specified minimum duration. The sequence and duration of each actuated phase are determined by traffic demand. Hence, this type of control is *not* associated with a constant cycle length.

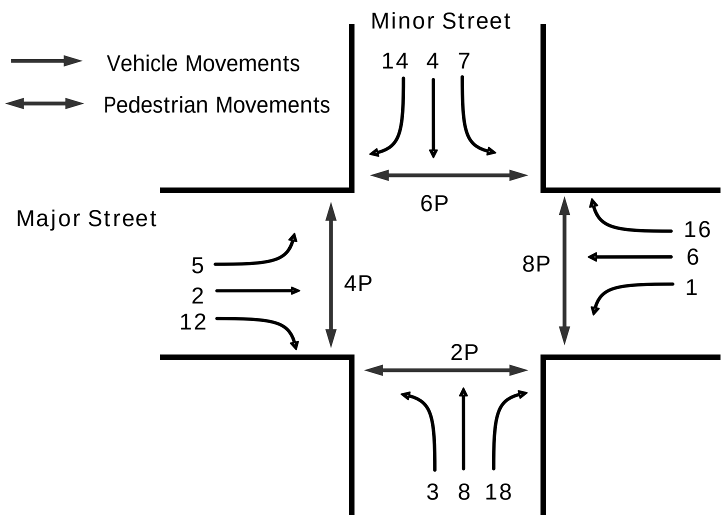
- *Coordinated-actuated control* is a variation of semiactuated operation. It uses the controller's force-off settings to constrain the noncoordinated phases associated with the minor movements so that the coordinated phases are served at the appropriate time during the signal cycle, and progression for the major movements is maintained. This type of control *is* associated with a constant cycle length.

Signalized intersections located close to one another on the same street are often operated as a coordinated signal system, in which specific phases at each intersection are operated on a common time schedule to permit the continuous flow of the associated movements at a planned speed. The signals in a coordinated system typically operate by using pretimed or coordinated-actuated control, and the coordinated phases typically serve the major-street through movements. Signalized intersections that are not part of a coordinated system are characterized as "isolated" and typically operate by using fully actuated or semiactuated control.

Intersection Traffic Movements

Exhibit 19-1 illustrates typical 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. Each movement is assigned a unique number or a number and letter combination. The letter "P" denotes a pedestrian movement. The number assigned to each left-turn and through movement is the same as the number assigned to each phase by National Electrical Manufacturers Association (NEMA) specifications.

Exhibit 19-1 Intersection Traffic Movements and Numbering Scheme



Intersection traffic movements are assigned the right-of-way by the signal controller. Each movement is assigned to one or more signal phases. A phase is defined as the green, yellow change, and red clearance intervals in a cycle that are assigned to a specified traffic movement (or movements) (*28*). The assignment of movements to phases varies in practice, depending on the desired phase sequence and the movements present at the intersection.

Signal Phase Sequence

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. Early controllers used a single-ring structure in which all nonconflicting movements were assigned to a common phase, and its duration was dictated by the movement needing the most time. Of the two structures, the dual-ring structure is more efficient because it allows the controller to adapt phase duration and sequence to the needs of the individual movements. The dual-ring structure is typically used with eight phases; however, more phases are available for complex signal phasing. The eight-phase dual-ring structure is shown in Exhibit 19-2. The symbol Φ represents the word *phase*, and the number following the symbol represents the phase number.

Exhibit 19-2
Dual-Ring Structure with Illustrative Movement Assignments

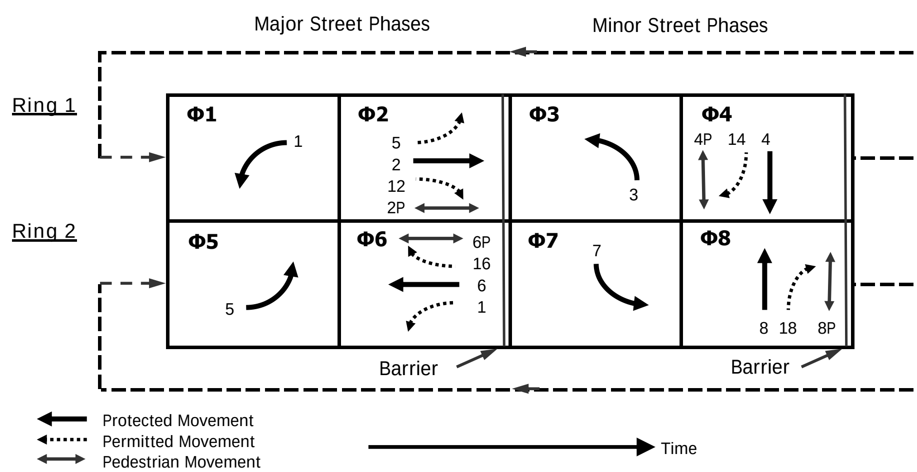


Exhibit 19-2 shows one way traffic movements can be assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is assigned to an exclusive phase. During this phase, the left-turn movement is "protected," so it receives a green arrow indication. 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.

Two rings and two barriers are identified in Exhibit 19-2. A ring consists of two or more sequentially timed conflicting phases. Ring 1 consists of Phases 1, 2, 3, and 4. Ring 2 consists of Phases 5, 6, 7, and 8. A barrier is used when there are two or more rings. It represents a reference point in the cycle at which one phase in each ring must reach a common point of termination. In Exhibit 19-2, a barrier is shown following Phases 2 and 6. A second barrier is shown following Phases 4 and 8. Between barriers, only one phase can be active at a time in each ring.

The ring structure dictates the sequence of phase presentation. Some common rules are provided in the following list:

- Phase Pairs 1–2, 3–4, 5–6, and 7–8 typically occur in sequence. Thus, Phase Pair 1–2 begins with Phase 1 and ends with Phase 2. Within each phase pair, it is possible to reverse the order of the pair. Thus, Pair 1–2 could be set to begin with Phase 2 and end with Phase 1 if it is desired to have the left-turn Phase 1 lag through Phase 2.
- Phase Pair 1–2 can operate concurrently with Phase Pair 5–6. That is, Phase 1 or 2 can time with Phase 5 or 6. Similarly, Phase Pair 3–4 can operate concurrently with Phase Pair 7–8. These phase pairs are also known as concurrency groups.
- For a given concurrency group, the last phase to occur in one phase pair must end at the same time as the last phase to occur in the other pair (i.e., they end together at the barrier).
- Phases between two barriers are typically assigned to the movements on a common street. For example, the four phases between the first and second barriers shown in Exhibit 19-2 are assigned to the minor street.

Operational Modes

There are three operational modes for the turn movements at an intersection. The names used to describe these modes refer to the way the turn movement is served by the controller. The three modes are as follows:

- Permitted,
- Protected, and
- Protected-permitted.

The *permitted mode* requires turning drivers to yield to conflicting traffic streams before completing the turn. Permitted left-turning drivers yield to oncoming vehicles and conflicting pedestrians. Permitted right-turning drivers yield to conflicting pedestrians. The efficiency of this mode depends on the availability of gaps in the conflicting streams. An exclusive turn lane may be provided, but it is not required. The permitted turn movement is typically presented with a circular green indication (although some agencies use other indications, such as a flashing yellow arrow). The right-turn movements in Exhibit 19-2 are operating in the permitted mode.

The *protected mode* gives turning drivers the right-of-way during the associated turn phase, while all conflicting movements are required to stop. This mode provides for efficient turn-movement service; however, the additional turn phase typically results in increased delay to the other movements. An exclusive turn lane is typically provided with this mode. The turn phase is indicated by a green arrow signal indication. Left-turn Movements 3 and 7 in Exhibit 19-2 are operating in the protected mode.

The *protected-permitted mode* represents a combination of the permitted and protected modes. Turning drivers have the right-of-way during the associated turn phase. Turning drives can also complete the turn “permissively” when the adjacent through movement receives its circular green (or when the turning driver receives a flashing yellow arrow) indication. This mode provides for efficient turn-movement service, often without causing a significant increase in the delay to other movements. Left-turn Movements 1 and 5 in Exhibit 19-2 are operating in the protected-permitted mode.

The operational mode used for one left-turn movement is often also used for the opposing left-turn movement. For example, if one left-turn movement is permitted, then so is the opposing left-turn movement. However, the modes for opposing left-turn movements are not required to be the same.

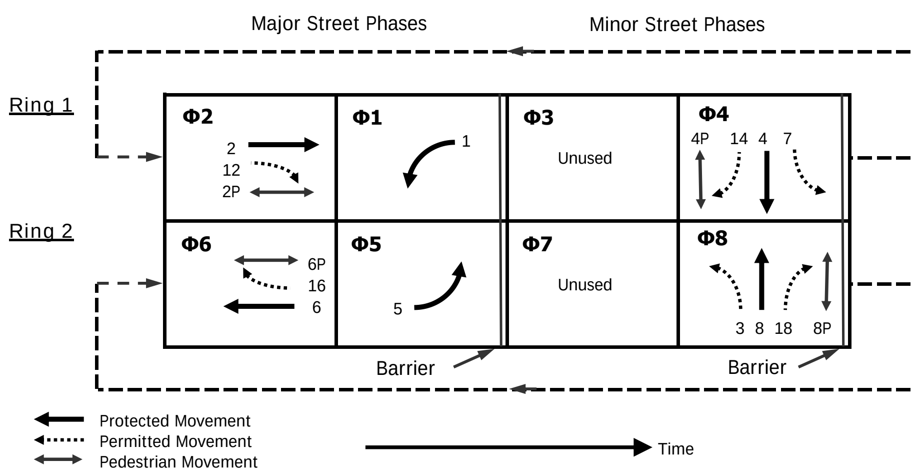
Left-Turn Phase Sequence

This subsection describes the sequence of service provided to left-turn movements relative to the other intersection movements. The typical options include the following:

- No left-turn phase (i.e., permitted only),
- Leading left-turn phase,
- Lagging left-turn phase, or
- Split phasing.

The permitted-only option is used when the left-turn movement operates in the permitted mode. A left-turn phase is not provided with this option. An illustrative implementation of permitted-only phasing for left- and right-turning traffic is shown in Exhibit 19-3 for the minor street.

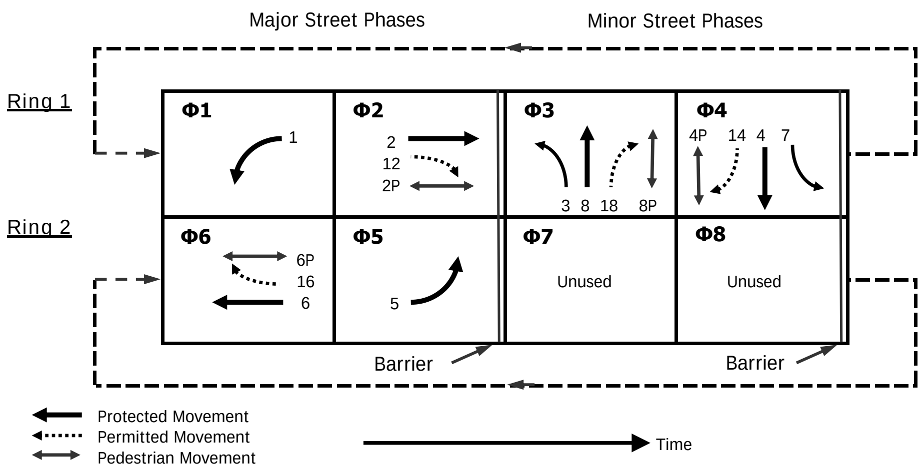
Exhibit 19-3
Illustrative Protected Lag–Lag and Permitted-Only Phasing



A leading, lagging, or split-phase sequence is used when the left turn operates in the protected mode or the protected-permitted mode. *Leading* and *lagging* indicate the order in which the left-turn phase is presented relative to the conflicting through movement. The leading left-turn sequence is shown in Exhibit 19-2 for the left-turn movements on the major and minor streets. The lagging left-turn sequence is shown in Exhibit 19-3 for the left-turn movements on the major street. A mix of leading and lagging phasing (called lead–lag) is shown in Exhibit 19-4 for the left-turn movements on the major street.

Split phasing describes a phase sequence in which one phase serves all movements on one approach and a second phase serves all movements on the opposing approach. Split phasing requires that all approach movements simultaneously receive a green indication. Split phasing is shown in Exhibit 19-4 for the minor street. Other variations of split phasing exist and depend on the treatment of the pedestrian movements. The left-turn movement in a split phase typically operates in the protected mode (as shown), provided there are no conflicting pedestrian movements.

Exhibit 19-4
Illustrative Protected Lead–Lag and Split Phasing



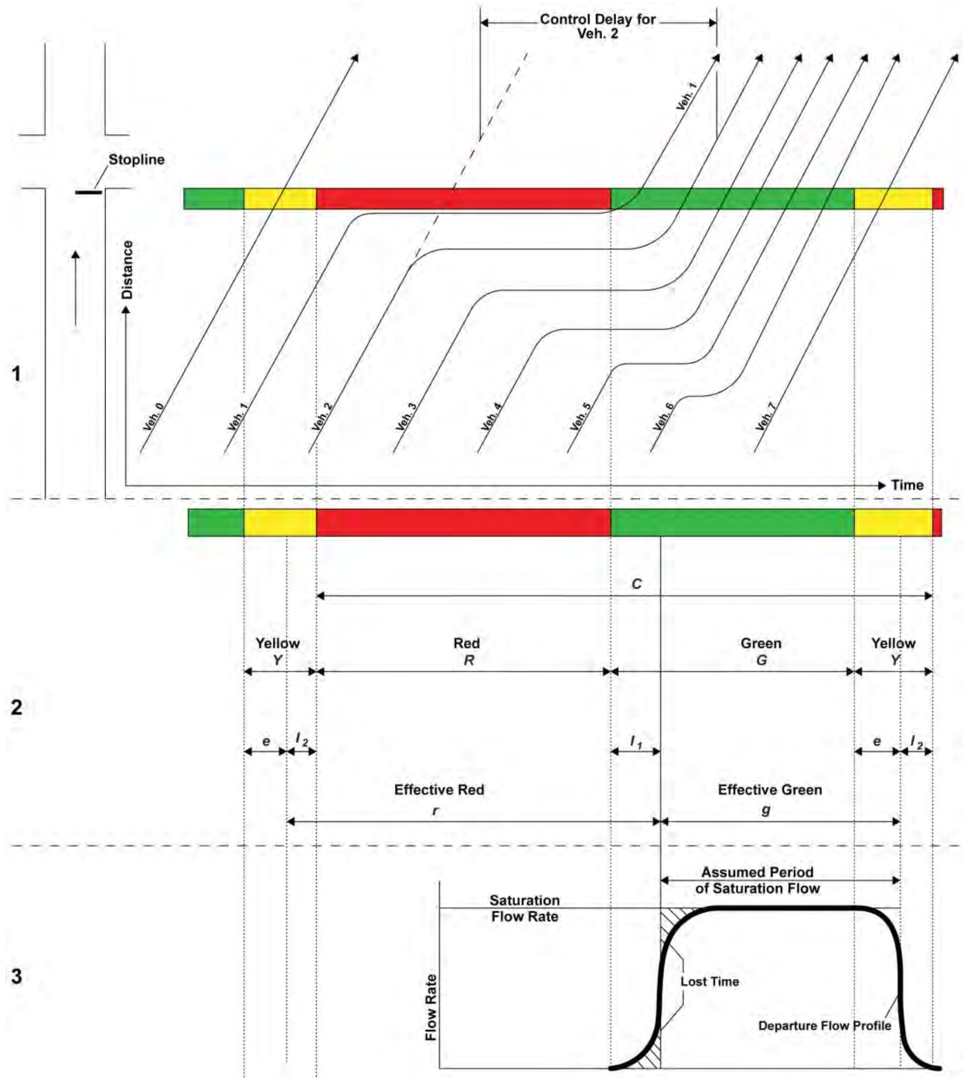
Traffic Flow Characteristics

This subsection describes several fundamental attributes of flow at signalized intersections. Exhibit 19-5 provides a reference for much of the discussion. The diagram represents a simple situation of vehicles on one approach to a signalized intersection during one signal cycle. The red clearance interval is not used in the example phase sequence shown in the exhibit.

Exhibit 19-5 is divided into three parts. Part 1 shows the time–space trajectory of several vehicles on the approach as they travel to (and through) the intersection. The horizontal bar represents the signal display (or “indication”) over time. It is located in the figure at a position that coincides with the intersection stop line. Part 2 shows the durations of the displayed red, green, and yellow intervals. It also shows the effective green, effective red, and lost time durations. The terms and symbols shown in Part 2 of Exhibit 19-5 are used throughout this chapter and Chapter 31, Signalized Intersections: Supplemental. Part 3 shows a flow profile diagram of the discharge flow rate (measured at the stop line) as a function of time.

The motorized vehicle methodology described in this chapter disaggregates the signal cycle into an effective green time and an effective red time for each phase to facilitate the evaluation of intersection operation. These two times are shown in Part 2 of Exhibit 19-5. Effective green time is the time that can be used by vehicles to proceed effectively at the saturation flow rate. Effective red time for a phase is equal to the cycle length minus the effective green time. Formal definitions for the effective green and red times are provided in Exhibit 19-6.

Exhibit 19-5
Fundamental Attributes of Traffic Flow at Signalized Intersections



Lost Time

As shown in Part 2 of Exhibit 19-5, two increments of lost time are associated with a phase. At the beginning of the phase, the first few vehicles in the queue depart at headways that exceed the saturation headway. The longer headway reflects the additional time the first few drivers require to respond to the change in signal indication and accelerate to the running speed. The start-up losses are called start-up lost time l_1.

At the end of the phase, the yellow indication is presented, and approaching drivers prepare for the signal to change to red. An initial portion of the yellow is consistently used by drivers and is referred to as the extension of the effective green *e*. The latter part of the change period (i.e., the yellow change interval and the red clearance interval), which is not used, is referred to as clearance lost time l_2. Phase lost time l_t equals the sum of the start-up and clearance lost times. Formal definitions for these terms are provided in Exhibit 19-6.

Exhibit 19-6
Fundamental Variables of Traffic Flow at Signalized Intersections

Term	Symbol	Definition
Terms Used as Variables		
Green interval duration (s)	G	The duration of the green interval associated with a phase. A green indication is displayed for this duration.
Yellow change interval (s)	Y	This interval follows the green interval. It is used to warn drivers of the impending red indication. A yellow indication is displayed for this duration.
Red clearance interval (s)	R_c	This interval follows the yellow change interval and is optionally used to provide additional time before conflicting movements receive a green indication.
Change period	CP	The sum of the yellow change interval and red clearance interval for a given phase.
Red time (s)	R	The time in the signal cycle during which the signal indication is red for a given phase.
Cycle length (s)	C	The total time for a signal to complete one cycle.
Effective green time (s)	g	The time during which a combination of traffic movements is considered to proceed effectively at the saturation flow rate.
Effective red time (s)	r	The time during which a combination of traffic movements is not considered to proceed effectively at the saturation flow rate. It is equal to the cycle length minus the effective green time.
Extension of effective green (s)	e	The initial portion of the yellow change interval during which a combination of traffic movements is considered to proceed effectively at the saturation flow rate.
Start-up lost time (s)	l_1	The additional time consumed by the first few vehicles in a queue whose headway exceeds the saturation headway because of the need to react to the initiation of the green interval and accelerate.
Clearance lost time (s)	l_2	The latter part of the change period that is not typically used by drivers to proceed through the intersection (i.e., they use this time to stop in advance of the stop line).
Phase lost time (s)	l_t	The sum of the clearance lost time and start-up lost time.
Cycle lost time (s)	L	The time lost during the cycle. It represents the sum of the lost time for each critical phase.
Adjusted saturation flow rate (veh/h/ln)	s	The equivalent hourly rate at which previously queued vehicles can traverse an intersection approach under prevailing conditions, assuming the green signal is available at all times and no lost times are experienced.
Control delay (s/veh)	d	The component of delay that results when a traffic control device causes a traffic movement to reduce speed or to stop. It represents the increase in travel time relative to the uncontrolled condition.
Terms Not Used as Variables		
Cycle		The time to complete one sequence of signal indications.
Interval		A period of time during which all signal indications remain constant.
Phase		The green, yellow change, and red clearance intervals assigned to a specified movement (or movements).

The relationship between phase lost time and signal timing is shown in Equation 19-1.

Equation 19-1

$$l_t = l_1 + l_2$$
$$l_t = l_1 + Y + R_c - e$$

where

l_t = phase lost time (s),

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

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

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

Y = yellow change interval (s), and

R_c = red clearance interval (s).

Research (*29*) has shown that start-up lost time is about 2 s and the extension of effective green is about 2 s (longer times may be appropriate for congested conditions, higher speeds, or heavy vehicles). If start-up lost time equals the extension of effective green, then phase lost time is equal to the change period (i.e., $l_t = Y + R_c$).

Saturation Flow Rate

Saturation flow rate is the equivalent hourly rate at which previously queued vehicles can traverse an intersection approach, assuming the green signal is available at all times and no lost times are experienced. It is expressed as an expected average hourly rate in units of vehicles per hour per lane. The concept of saturation flow rate is discussed in more detail in Chapter 4, Traffic Operations and Capacity Concepts.

The *base* saturation flow rate represents the expected average flow rate for a through-traffic lane for exceptionally favorable geometric and traffic conditions (e.g., no grade, no trucks, and so forth). The *adjusted* saturation flow rate represents the saturation flow rate for prevailing geometric and traffic conditions. Prevailing conditions typically result in the adjusted saturation flow rate being smaller than the base saturation flow rate.

A procedure for estimating the adjusted saturation flow rate for a lane group is provided in Section 3. The procedure consists of a base saturation flow rate and a series of adjustment factors. The factors are used to adjust the base rate to reflect the prevailing conditions associated with the subject lane group.

The saturation flow rate for prevailing conditions can be determined directly from field measurement. A technique for measuring this rate is described in Section 5 of Chapter 31, Signalized Intersections: Supplemental.

Capacity

Capacity is defined as the maximum number of vehicles that can reasonably be expected to pass through the intersection under prevailing traffic, roadway, and signalization conditions during a 15-min period. Capacity is computed as the product of adjusted saturation flow rate and effective green–to–cycle length ratio. Capacity is expressed as an expected average hourly rate in units of vehicles per hour.

Phase Duration

This subsection describes the components of phase duration. The discussion is focused on an actuated phase; however, some elements of the discussion are equally applicable to a pretimed phase.

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 19-2.

Equation 19-2

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

where

D_p = phase duration (s),

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

g_s = queue service time (s),

g_e = green extension time (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

The relationship between the variables in Equation 19-2 is shown in Exhibit 19-7 by using a queue accumulation polygon.

Exhibit 19-7
Time Elements Influencing Actuated Phase Duration

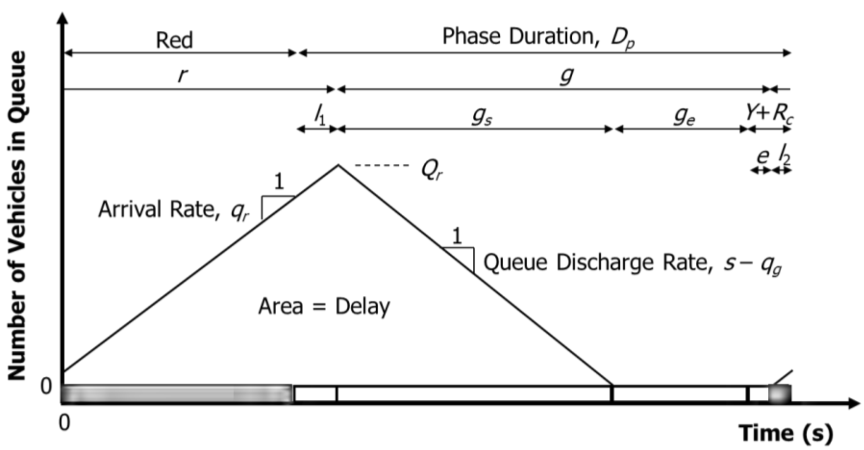


Exhibit 19-7 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the red interval ends. At this time, the queue begins to discharge at a rate equal to the saturation flow rate s less the arrival rate during green q_g. The queue clears g_s seconds after it first begins to discharge. Thereafter, random vehicle arrivals are detected and cause the green interval to be extended (provided the headway between vehicles remains below a specified value). Eventually, a gap occurs in traffic (or the maximum green limit is reached), and the green interval ends. The end of the green interval coincides with the end of the extension time g_e.

The effective green time for the phase is computed as shown in Equation 19-3.

Equation 19-3

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

where

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

e = extension of effective green = 2.0 (s).

ANALYSIS TYPE

The term *analysis type* is used to describe the purpose for which a methodology is used. Each purpose is associated with a different level of detail as it relates to the precision of the input data, the number of default values used, and the desired accuracy of the results. Three analysis types are recognized in this chapter:

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

These analysis types are discussed in more detail in Chapter 2, Applications.

SPATIAL AND TEMPORAL LIMITS

Analysis Boundaries

The intersection analysis boundary is defined by the operational influence area of the intersection. It is not defined as a fixed distance for all intersections. Rather, it extends upstream from the intersection on each intersection leg. The size of this area is leg specific. It includes all geometric features and traffic conditions that influence intersection operation during the study period. For these reasons, the analysis boundaries should be established for each intersection on the basis of the conditions present during the study period.

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 duration of the analysis period is in the range of 0.25 to 1 h. The longer durations in this range are sometimes used for planning analyses. In general, the analyst should use caution with analysis periods that exceed 1 h both because traffic conditions typically are not steady for long time periods and because the adverse impact of short peaks in traffic demand may not be detected in the evaluation.

LOS CRITERIA

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

Motorized Vehicle Mode

All uses of the terms volume *or* volume-to-capacity ratio *in this chapter refer to demand volume or demand volume-to-capacity ratio.*

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

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

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

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

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

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

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

A lane group can incur a delay less than 80 s/veh when the volume-to-capacity ratio exceeds 1.0. This condition typically occurs when the cycle length is short, the signal progression is favorable, or both. As a result, both the delay and volume-to-capacity ratio are considered when lane group LOS is established. A ratio of 1.0 or more indicates 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 19-8 lists the LOS thresholds established for the motorized vehicle mode at a signalized intersection.

Exhibit 19-8
LOS Criteria: Motorized Vehicle 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.

Pedestrian and Bicycle Modes

Historically, the HCM has used a single performance measure as the basis for defining LOS. However, research documented in Chapter 5, Quality and Level-of-Service Concepts, indicates travelers consider a wide variety of factors in assessing the quality of service provided to them. Some of these factors can be described as performance measures (e.g., speed) and others can be described as basic descriptors of the intersection character (e.g., crosswalk width). The methodologies for evaluating the pedestrian and bicycle modes combine these factors to determine the corresponding mode's LOS.

Exhibit 19-9 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 19-9
LOS Criteria: Pedestrian and Bicycle Modes

LOS	LOS Score
A	≤1.50
B	>1.50–2.50
C	>2.50–3.50
D	>3.50–4.50
E	>4.50–5.50
F	>5.50

SCOPE OF THE METHODOLOGIES

This subsection identifies the conditions for which each methodology is applicable.

- *Intersection geometry*. All methodologies apply to three- and four-leg intersections of two streets or highways.
- *Flow conditions*. The three methodologies are based on the analysis of steady traffic conditions and, as such, are not well suited to the evaluation of unsteady conditions (e.g., congestion, queue spillback, signal preemption).

- *Target road users*. Collectively, the three methodologies were developed to estimate the LOS perceived by motorized vehicle drivers, pedestrians, and bicyclists. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers, automobile passengers, delivery truck drivers, recreational vehicle drivers). However, it is likely the perceptions of these other road users are reasonably well represented by the road users for whom the methodologies were developed.
- *Influences in the right-of-way*. A road user's perception of quality of service is influenced by many factors inside and outside the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside the right-of-way (e.g., buildings, parking lots, scenery, landscaped yards) that might influence a traveler's perspective. This approach was followed because factors outside the right-of-way are not under the direct control of the agency operating the street.

3. CORE MOTORIZED VEHICLE METHODOLOGY

This section describes the methodology for evaluating the capacity and quality of service provided to motorized vehicles at a signalized intersection. Basic extensions of this methodology to address critical intersection volume-to-capacity ratio and initial queue presence are provided in Section 4. Extensions to this methodology for evaluating more complex intersection operational elements (e.g., queue length) are described in Chapter 31, Signalized Intersections: Supplemental.

SCOPE OF THE METHODOLOGY

The overall scope of the three methodologies is provided in Section 2. This section identifies the additional conditions for which the motorized vehicle methodology is applicable.

- *Upstream intersections*. The influence of an upstream signalized intersection on the subject intersection's operation is addressed by input variables that simply, and subjectively, describe platoon structure and the uniformity of arrivals on a cyclic basis. Chapter 18, Urban Street Segments, extends the methodology described in this chapter to the evaluation of an intersection that is part of a coordinated signal system.
- *Target travel modes*. The motorized vehicle methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The methodology is not designed to evaluate the performance of other types of vehicles (e.g., golf carts, motorized bicycles).

Spatial and Temporal Limits

Analysis Boundaries

The intersection analysis boundary is defined by the operational influence area of the intersection. It should include the most distant extent of any intersection-related queue expected to occur during the study period. For these reasons, the influence area is likely to extend at least 250 ft back from the stop line on each intersection leg.

Study Period and Analysis Period

The concepts of *study period* and *analysis period* are defined in Section 2 with general terms. They are defined more precisely in this subsection as they relate to the motorized vehicle methodology.

Exhibit 19-10 demonstrates three alternative approaches an analyst might use for a given evaluation. Other alternatives exist, and the study period can exceed 1 h. Approach A is the approach that has traditionally been used and, unless otherwise justified, is the approach that is recommended for use.

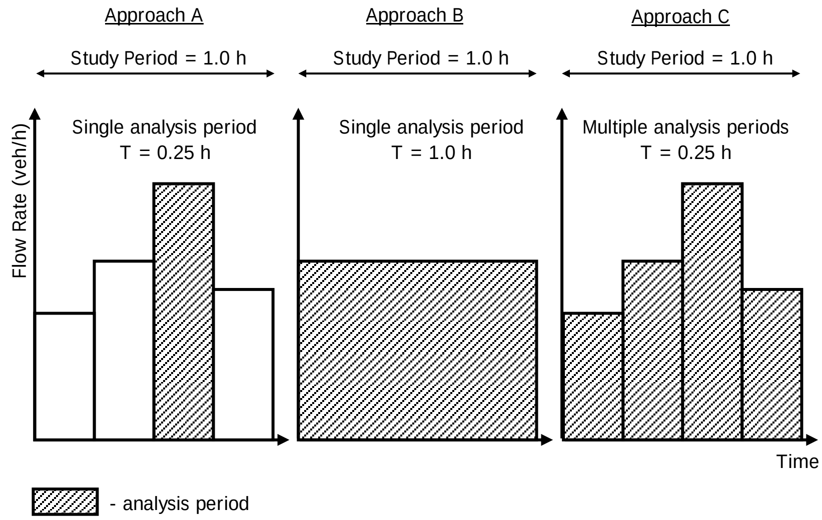


Exhibit 19-10
Three Alternative Study Approaches

The use of a peak 15-min traffic count multiplied by four is preferred for existing conditions when traffic counts are available. The use of a 1-h demand volume divided by a peak hour factor is preferred when projected volumes are used or when volumes are used that have been added to existing volumes.

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 used for the analysis is based on either (*a*) a peak 15-min traffic count multiplied by four or (*b*) a 1-h demand volume divided by the peak hour factor. The former option is preferred for existing conditions when traffic counts are available; the latter option is preferred when hourly projected volumes are used or when hourly projected volumes are added to existing volumes. Additional discussion on use of the peak hour factor is provided in the subsection titled Required Data and Sources.

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

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

Regardless of analysis period duration, a single-period analysis (i.e., Approach A or B) is typical for planning applications.

Multiple Time-Period Analysis

If the analysis period's demand volume exceeds capacity, then a multiple time-period analysis should be undertaken when the study period includes an initial analysis period with no initial queue and a final analysis period with no residual queue. The initial queue for the second and subsequent periods is equal to the residual queue from the previous period. 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 separately reported. 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 in reality some analysis periods have unacceptable operation.

Performance Measures

Performance measures applicable to the motorized vehicle travel mode include volume-to-capacity ratio, control delay, and queue storage ratio. The queue storage ratio describes the ratio of the back-of-queue size to the available vehicle storage length. The back of queue represents the maximum backward extent of queued vehicles during a typical cycle.

LOS is also considered a performance measure. It is useful for describing intersection performance to elected officials, policy makers, administrators, or the public. LOS is based on control delay.

Limitations of the Methodology

This subsection identifies the known limitations of the motorized vehicle methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific street segment, then the analyst should consider using alternative methods or tools for the evaluation.

The motorized vehicle methodology does not account for the effect of the following conditions on intersection operation:

- Turn bay overflow;
- Multiple advance detectors in the same lane;
- Demand starvation due to a closely spaced upstream intersection;
- Queue spillback into the subject intersection from a downstream intersection;
- Queue spillback from the subject intersection into an upstream intersection;
- Premature phase termination due to short detection length, passage time, or both;
- Right-turn-on-red (RTOR) volume prediction or resulting right-turn delay;
- Turn movements served by more than two exclusive lanes;
- Delay to traffic movements that are not under signal control;
- Through lane (or lanes) added just upstream of the intersection or dropped just downstream of the intersection; and
- Storage of shared-lane left-turning vehicles within the intersection to permit bypass by through vehicles in the same lane.

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

- Rest-in-walk mode for actuated and noncoordinated phases,
- Preemption or priority modes,
- Phase overlap (see discussion that follows), and
- Gap reduction or variable initial settings for actuated phases.

Two control strategies that use phase overlap are addressed by the methodology. One strategy is "right-turn overlap with the complementary left-turn phase." This strategy uses the overlap feature to provide a protected signal indication for the right-turn movement that is concurrent with the complementary left-turn phase. Procedures for evaluating this operation are described in Chapter 31, Signalized Intersections: Supplemental. They address the case in which the right-turn movement has an exclusive turn lane.

The second strategy, "left-turn overlap with the opposing through phase," uses the overlap feature to provide the signal indication for permissive left-turn operation that is concurrent with the opposing through phase. This overlap feature is referred to as "Dallas left-turn phasing." It is typically used with flashing yellow operation. Procedures for evaluating this operation are described in Chapter 31.

Lane Groups and Movement Groups

The motorized vehicle methodology uses the concepts of *lane groups* and *movement 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. Guidelines for establishing lane groups and movement groups are described in the subsection titled Computational Steps.

Lane Groups

The motorized vehicle methodology is designed to analyze 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. A lane group can include one or more lanes.

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

Movement Groups

The concept of *movement groups* is established to facilitate data entry to the methodology. In this regard, input data describing intersection traffic are traditionally specific to the movement (e.g., left-turn movement volume); the data are not specific to the lane (e.g., analysts rarely have the volume for a lane shared by left-turning and through vehicles). Thus, most traffic characteristic–related and geometric design–related input data are specific to a movement group.

The basic principle for establishing movement groups is that no traffic movement can be assigned to more than one movement group. Thus, 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). A movement group can include one or more lanes.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the motorized vehicle methodology. These data are listed in Exhibit 19-11. The additional data needed for coordinated control are listed in Exhibit 19-12. The second column (labeled Basis) of both Exhibit 19-11 and Exhibit 19-12 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 exhibits also list potential data sources and default values that can be used if local data are not available (*30*).

The data elements listed in Exhibit 19-11 and Exhibit 19-12 do not include variables that are considered to represent calibration factors (e.g., start-up lost time). A calibration factor typically has a relatively narrow range of reasonable values or has 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.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 19-11. These data describe the motorized vehicle traffic stream that travels through the intersection during the study period.

Demand Flow Rate

The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. Demand flow rate represents the flow rate of vehicles *arriving* at the intersection. The "count of vehicles" can be obtained from a variety of sources (e.g., from the field or as a forecast from a planning model).

When measured in the field, the demand 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 estimate that is lower than the true rate (i.e., the estimate will equal capacity rather than true demand).

Exhibit 19-11
Required Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
Traffic Characteristics			
Demand flow rate (veh/h)	M	Field data, past counts	Must be provided
Right-turn-on-red flow rate (veh/h)	A	Field data, past counts	0.0 veh/h
Percentage heavy vehicles (%)	MG	Field data, past counts	3%
Peak hour factor (decimal)	I	Field data, analyst judgment	Hourly data and 0.25-h analysis period: Total entering vol. ≥1,000 veh/h: 0.92 Total entering vol. <1,000 veh/h: 0.90 Otherwise: 1.00
Platoon ratio (decimal)	MG	Field data, analyst judgment	See discussion
Upstream filtering adjustment factor (decimal)	MG	Field data, analyst judgment	1.0
Initial queue (veh)	MG	Field data, analyst judgment	Must be provided
Base saturation flow rate (pc/h/ln)	MG	Field data, analyst judgment	Metro pop. ≥250,000: 1,900 pc/h/ln Otherwise: 1,750 pc/h/ln
Lane utilization adjustment factor (decimal)	MG	Field data, analyst judgment	See discussion
Pedestrian flow rate (p/h)	A	Field data, past counts	Must be provided
Bicycle flow rate (bicycles/h)	A	Field data, past counts	Must be provided
On-street parking maneuver rate (veh/h)	MG	Field data, analyst judgment	See discussion
Local bus stopping rate (buses/h)	A	Field data, analyst judgment	CBD bus stop: 12 buses/h Non-CBD bus stop: 2 buses/h
Unsignalized movement delay (s)	M	Field data	See discussion
Geometric Design			
Number of lanes (ln)	M	Field data, aerial photo	Must be provided
Average lane width (ft)	MG	Field data, aerial photo	12 ft
Number of receiving lanes (ln)	A	Field data, aerial photo	Must be provided
Turn bay length (ft)	MG	Field data, aerial photo	Must be provided
Presence of on-street parking	MG	Field data, aerial photo	Must be provided
Approach grade (%)	A	Field data	Flat approach: 0% Moderate grade on approach: 3% Steep grade on approach: 6%
Signal Control			
Type of signal control	I	Field data	Must be provided
Phase sequence	A	Field data	Must be provided
Left-turn operational mode	A	Field data	Must be provided
Dallas left-turn phasing option	A	Field data	Dictated by local use
Passage time (s) (if actuated)	P	Field data	2.0 s (presence detection)
Maximum green (s) (if actuated)	P	Field data	Major-street through movement: 50 s Minor-street through movement: 30 s Left-turn movement: 20 s
Green duration (s) (if pretimed)	P	Field data	Major-street through movement: 50 s Minor-street through movement: 30 s Left-turn movement: 20 s
Minimum green (s)	P	Field data	Major-street through movement: 10 s Minor-street through movement: 8 s Left-turn movement: 6 s
Yellow change + red clearance (s)[a]	P	Field data	4.0 s
Walk (s)	P	Field data	Actuated: 7.0 s Pretimed: green interval minus pedestrian clear
Pedestrian clear (s)	P	Field data	Based on 3.5-ft/s walking speed
Phase recall (if actuated)	P	Field data	No recall
Dual entry (if actuated)	P	Field data	Not enabled (i.e., use single entry)
Simultaneous gap-out (if actuated)	A	Field data	Enabled
Cycle length (if pretimed)	I	Field data	See discussion

Note: Exhibit continues on the next page.

Exhibit 19-11 **(cont'd.)**
Required Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
		Other Data	
Analysis period duration (h)	I	Set by analyst	0.25 h
Speed limit (mi/h)	A	Field data, road inventory	Must be provided
Stop-line detector length (ft) and detection mode (if actuated)	MG	Field data	40 ft, presence detection mode
Area type (CBD, non-CBD)	I	Analyst judgment	Must be provided

Notes: M = movement: one value for each left-turn, through, and right-turn movement.
A = approach: one value or condition for the intersection approach.
MG = 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).
I = intersection: one value or condition for the intersection.
P = phase: one value or condition for each signal phase.
CBD = central business district; vol. = volume; pop. = population.
[a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

Exhibit 19-12
Required Additional Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis with Coordinated Signal Control

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
Cycle length (s)	I	Field data	See discussion
Phase splits (s) (if actuated)	P	Field data	See discussion
Offset (s)	I	Field data	Equal to travel time in Phase 2 direction[a]
Offset reference point (s)	I	Field data	End of green for Phase 2[a]
Force mode (if actuated)	I	Field data	Fixed

Notes: I = intersection: one value or condition for the intersection.
P = phase: one value or condition for each signal phase.
[a] Assumes Phase 2 is the reference phase. Substitute 6 if Phase 6 is the reference phase.

If a planning analysis is being conducted in which (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand should 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 in Exhibit 19-11 can be used.

If a multiple time-period analysis is conducted (i.e., Approach C in Exhibit 19-10), then the intersection's demand flow rates should be provided for each analysis period.

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. This assumption may not hold for situations in which drivers choose a lane so they are prepositioned for a turn at the downstream intersection. Similarly, it may not hold when an auxiliary through lane is present. In either situation, the analyst will need to provide the demand flow rate for each lane on the approach and then combine these rates to define the demand flow rate for each lane group. Additional discussion of this topic is provided in the subsection titled Lane Utilization Adjustment Factor.

The demand flow rate for all signal-controlled movements must be provided. The demand flow rate for all unsignalized movements should be provided. If an unsignalized movement exists but its flow rate is not provided, then this movement will be excluded from the calculation of approach delay and intersection delay.

Right-Turn-on-Red Flow Rate

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

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

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

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

Percentage 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 Equation 19-4.

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

Equation 19-4

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 19-4 must be taken during a 15-min period that occurs within the 1-h period represented by the variable in the numerator. Both variables in this equation represent the total number of vehicles entering the intersection during their respective time period. As such, one peak hour factor is computed for the intersection. This factor is then applied individually to each traffic movement. Values of this factor typically range from 0.80 to 0.95.

The peak hour factor is used primarily for a planning analysis when a forecast hourly volume is provided and an analysis of the peak 15-min period is sought.

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

If the peak hour factor is used, a single intersectionwide factor should be used rather than movement-specific or approach-specific factors. If individual approaches or movements peak at different times, a series of 15-min analysis periods that encompass the peaking should be considered.

If peak hour factors are used, a single peak hour factor for the entire intersection is generally preferred because it will decrease the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, but in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. In the event individual approaches or movements are known to have substantially different peaking characteristics or peak during different 15-min periods within the hour, a series of 15-min analysis periods that encompass the peaking should be considered instead of a single analysis period using a single peak hour factor for the intersection.

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 19-13 provides an indication of the quality of progression associated with selected platoon ratio values.

Exhibit 19-13
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 by using Equation 19-5.

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

Equation 19-5

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).

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.

Determining Platoon Ratio

If the subject intersection is part of a signal system, then the procedure in Section 3 of Chapter 30, Urban Street Segments: Supplemental, can be used to estimate the arrival flow profile for any approach that is evaluated as part of an urban street segment. The procedure uses the flow profile to compute the proportion of arrivals during the green indication. If this procedure is used, then platoon ratio is not an input for the traffic movements on the subject approach.

If the subject intersection is not part of a signal system and an existing intersection is being evaluated, then it is recommended that analysts use field-measured values for the variables in Equation 19-5 in estimating the platoon ratio.

If the subject intersection is not part of a signal system and the analysis deals with future conditions, or if the variables in Equation 19-5 are not known from field data, then the platoon ratio can be estimated by using guidance provided in the next subsection.

Guidance for Estimating Platoon Ratio

This subsection provides guidance for estimating arrival type. The platoon ratio can then be determined from this arrival type by using Exhibit 19-13.

Exhibit 19-14 provides guidance for selecting the arrival type for through movements. The guidance considers typical signal spacing and the provision of signal coordination. Arrival Type 3 is typically assumed for turn movements.

Exhibit 19-14
Arrival Type Selection Guidelines

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

A description of each arrival type is provided in the following paragraphs to provide additional assistance with the selection of arrival type.

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

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

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

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

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

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

Upstream Filtering Adjustment Factor

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

The filtering adjustment factor varies from 0.09 to 1.0. A value of 1.0 is appropriate for an isolated intersection (i.e., one that is 0.6 mi or more from the nearest upstream signalized intersection). A value of less than 1.0 is appropriate for nonisolated intersections. Equation 19-6 is used to compute I for nonisolated intersections.

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

Equation 19-6

where

I = upstream filtering adjustment factor, and

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

The variable X_u is computed as the weighted volume-to-capacity ratio of all upstream movements contributing to the volume in the subject movement group. This ratio is computed as a weighted average with the volume-to-capacity ratio of each contributing upstream movement weighted by its discharge volume. For planning and design analyses, X_u can be approximated as the volume-to-capacity ratio of the contributing through movement at the upstream signalized intersection. The value of X_u used in Equation 19-6 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.

Initial queue has a significant effect on delay and can vary widely among intersections and traffic movements. If it is not possible to obtain an initial queue estimate, then the analysis period should be established so the previous period is known to have demand less than capacity and no residual queue.

Base Saturation Flow Rate

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

Lane Utilization Adjustment Factor

The lane utilization adjustment factor accounts for the unequal distribution of traffic among the lanes in those movement groups with more than one exclusive lane. This factor provides an adjustment to the base saturation flow rate to account for uneven use of the lanes. It is not used unless a movement group has more than one exclusive lane. The lane utilization adjustment factor is calculated with Equation 19-7.

Equation 19-7

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

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 19-7 to establish local default values of the lane utilization adjustment factor.

A lane utilization factor of 1.0 is used when a movement group has only one lane or a uniform traffic distribution can be assumed across all exclusive lanes in the movement group. 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 downstream. 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 (in which case a lane utilization factor of 1.0 is used).

Some intersections have an auxiliary through lane (i.e., a through-lane addition on the approach to an intersection combined with a through-lane drop exiting the intersection). This type of lane can be underutilized if it is relatively short. When it is present on an approach, a more accurate evaluation will be obtained when the actual flow rate for each approach lane is provided as input to the methodology. A procedure for estimating approach lane volumes for this situation is provided in NCHRP Report 707 (*31*).

Default Value. The default lane utilization factors described in this subsection apply to situations in which drivers randomly choose among the exclusive-use lanes on the intersection approach. The factors do not apply to special conditions (such as short lane drops or a downstream freeway on-ramp) that might cause drivers intentionally to choose their lane position on the basis of an anticipated downstream maneuver. Exhibit 19-15 provides default lane utilization adjustment factors for different movement groups and numbers of lanes.

Exhibit 19-15
Default Lane Utilization Adjustment Factors

Movement Group	No. of Lanes in Movement Group (ln)	Traffic in Most Heavily Traveled Lane (%)	Lane Utilization Adjustment Factor f_{LU}
Exclusive through	1	100.0	1.000
	2	52.5	0.952
	3[a]	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2[a]	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2[a]	56.5	0.885

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

As demand approaches capacity, the analyst may use lane utilization factors that are closer to 1.0 than those offered in Exhibit 19-15. This refinement to the factor value recognizes that a high volume-to-capacity ratio is associated with a more uniform use of the available lanes because of reduced opportunity for drivers to select their lane freely.

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 motorized vehicle 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.

Exhibit 19-16 gives default values for the parking maneuver rate on an intersection approach with on-street parking. It is estimated for a distance of 250 ft back from the stop line. The calculations assume 25 ft per parking space and 80% occupancy. Each turnover (one car leaving and one car arriving) generates two parking maneuvers.

Exhibit 19-16
Default Parking Maneuver Rate

Street Type	No. of Parking Spaces in 250 ft	Parking Time Limit (h)	Turnover Rate (veh/h)	Maneuver Rate (maneuvers/h)
Two-way	10	1	1.0	16
		2	0.5	8
One-way	20	1	1.0	32
		2	0.5	16

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.

Unsignalized Movement Delay

The delay for unsignalized movements at the intersection should be provided as input to the methodology whenever these movements exist at the intersection. If provided, they can be included in the calculation of approach delay and intersection delay.

The delay will need to be estimated by means external to the methodology. These external means may include direct field measurement, observation of similar conditions, special application of procedures in other HCM chapters, and simulation. The level of effort expended for such estimation should be commensurate with the relevance the unsignalized delay has to the overall analysis. For example, high-volume or high-delay movements should be estimated carefully. Free-flow right turns can be assumed to have zero delay.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 19-11. These data describe the geometric elements of the intersection that influence traffic operation.

Number of Lanes

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

The number of lanes on an approach depends on approach volume and signal timing. A single exclusive left-turn lane is often provided when the left-turn volume ranges between 100 and 300 veh/h. Similarly, a dual exclusive left-turn lane is often provided when the left-turn volume exceeds 300 veh/h. An exclusive right-turn lane is often provided when the right-turn volume exceeds 300 veh/h and the adjacent through volume exceeds 300 veh/h/ln.

Average Lane Width

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

Number of Receiving Lanes

The number of receiving lanes represents the count of lanes departing the intersection. This number should be separately determined for each left-turn and right-turn movement. Experience indicates 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 can be stored in the two-way left-turn lane.

Presence of On-Street Parking

The input for presence of on-street parking 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 subsection describes the signal control data listed in Exhibit 19-11 and Exhibit 19-12. They are specific to an actuated signal controller that is operated in a pretimed, semiactuated, fully actuated, or coordinated-actuated manner.

Type of Signal Control

The intersection signal control is an input to the methodology. It can be pretimed control or actuated control. Pretimed control can be described as coordinated (or coordinated-pretimed) if the intersection is part of a signal system. Actuated control can be described as fully actuated, semiactuated, or coordinated-actuated.

Settings used for coordinated-actuated control are described later in this subsection. They are used in the motorized vehicle methodology in Chapter 18.

The motorized vehicle methodology is based on the controller functions defined in the National Transportation Communications for ITS Protocol Standard 1202 (*28*). It is incumbent on the analyst to become familiar with these functions and adapt them, if needed, to the functionality of the controller used at the subject intersection. Section 2 provides additional information about traffic signal controller operation.

Phase Sequence

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

Left-Turn Operational Mode

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

Dallas Left-Turn Phasing Option

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

Passage Time: Actuated Control

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, vehicle length, and vehicle speed. Longer passage times are often used with shorter detection zones, greater distance between the zone and stop line, fewer lanes, smaller vehicles, and higher speeds.

The objective in determining the passage time value is to make it large enough to ensure all queued vehicles are served but not so large that it extends for traffic arriving randomly after the queue has cleared. 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., max-out) so that safe phase termination is compromised.

Maximum Green: Actuated Control

The maximum green setting defines the maximum amount of time 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 values for through phases serving the major-street approach range from 30 to 60 s.

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.

Green Duration: Pretimed Control

For an analysis of pretimed operation, the green interval duration is an input to the methodology. Typical values are similar to those noted above for the maximum green setting. A procedure for estimating pretimed green interval duration is described in Section 2 of Chapter 31, Signalized Intersections: Supplemental.

Minimum Green

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

Yellow Change and Red Clearance

The yellow change and red clearance settings are input for each signal phase. The yellow change interval is intended to alert a driver to the impending presentation of a red indication. It ranges from 3 to 6 s, with longer values in this range used with phases serving high-speed movements.

The red clearance interval can be used to allow a brief time to elapse after the yellow indication, during which the signal heads associated with the ending phase and all conflicting phases display a red indication. If used, the red clearance interval is typically 1 or 2 s.

Walk

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

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

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

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

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

Pedestrian Clear

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

Phase Recall: Actuated Control

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

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

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

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

Dual Entry: Actuated Control

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

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

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

Simultaneous Gap-Out: Actuated Control

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

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

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

Cycle Length: Coordinated-Actuated or Pretimed Control

Cycle length is the time elapsed between the endings of two sequential presentations of a coordinated-phase green interval. A cycle length is needed for pretimed control and for coordinated-actuated control.

Default Value. The cycle length used for a coordinated signal system often represents a compromise value based on intersection capacity, queue size, phase sequence, segment length, speed, and progression quality. Consideration of these factors leads to the default cycle lengths shown in Exhibit 19-17.

Exhibit 19-17
Default System Cycle Length

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

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

Phase Splits: Coordinated-Actuated Control

Each noncoordinated phase is provided a “split” time that represents the sum of the green, yellow change, and red clearance intervals for the phase. The rationale for determining the green interval duration varies among agencies; however, it is often related to the “optimum” pretimed green interval duration. Section 2 of Chapter 31 provides a procedure for determining pretimed phase duration.

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

Offset and Offset Reference Point: Coordinated Control

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

Force Mode: Coordinated-Actuated Control

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

Other Data

This subsection describes the data listed in Exhibit 19-11 that are categorized as "other" data.

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. Its duration ranges from 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should use caution in interpreting the results from an analysis period of 1 h or more because the adverse impact of short peaks in traffic demand may not be detected.

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

Operational analyses require a 15-min analysis period. This duration will accurately capture the adverse effects of demand peaks.

Most planning analyses use a 15-min analysis period. However, a 1-h analysis period can be used, if appropriate.

Speed Limit

The methodology is based on the assumption that the posted speed limit is (*a*) consistent with posted speed limits 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 the posted speed limit does not satisfy these assumptions, then the speed limit value that is input to the methodology should be adjusted so it is consistent with the assumptions.

Stop-Line Detector Length and Detection Mode: Actuated Control

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

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

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

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

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

Area Type

The area type input indicates whether an 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 CBD-like characteristics is significantly longer than at intersections in areas that are less constrained and less visually intense.

OVERVIEW OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of the signalized intersection in terms of its service to motorized vehicles. The methodology is computationally intense and is most efficiently implemented using software. The intensity stems partly from the need to model traffic-actuated signal operation.

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

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

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

Framework

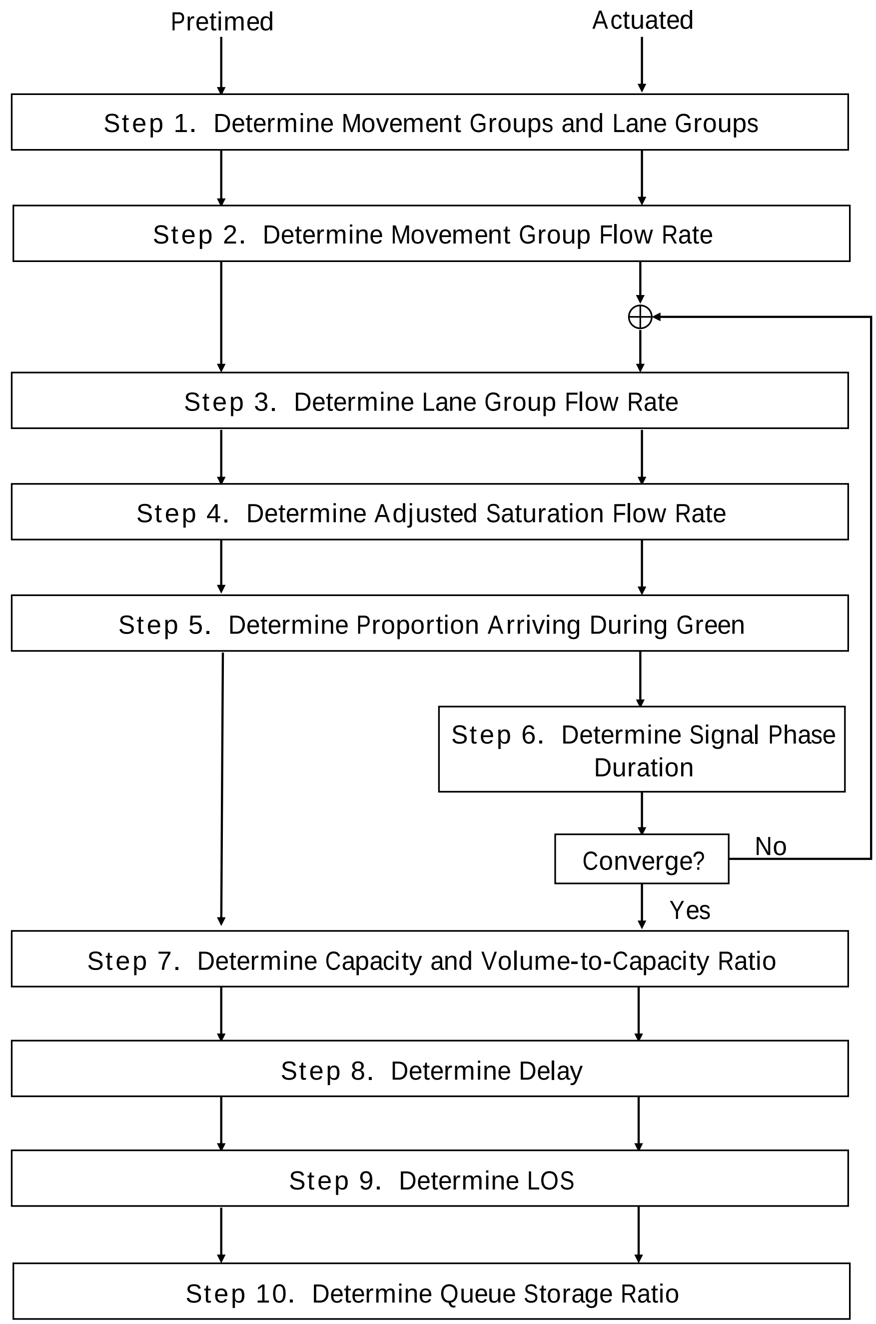
Exhibit 19-18 illustrates the calculation framework of the motorized vehicle methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. These calculations are described more fully in the next section.

The methodology adopts the same principles of critical movement analysis used in prior editions of the HCM. The first step of the methodology is used to determine the lane groups associated with each intersection approach. These lane groups represent the basic unit of analysis. Each lane group is separately evaluated, and the results are aggregated to the approach and intersection levels. The second and third steps are used to determine how the left-turn, through, and right-turn drivers on each intersection approach distribute themselves among the lane groups. The fourth step is used to predict the saturation flow rate for each lane group based on prevailing conditions. The fifth step is used to quantify the effect of upstream signals on the arrival flow rate for each lane group. If a phase is actuated, the sixth step is needed to estimate the average duration of this phase. In the seventh step, lane group capacity is evaluated in terms of the ratio of flow rate to capacity. This ratio is used in Step 8 to estimate the control delay for each lane group. This estimated control delay is used in Step 9 to estimate the LOS for each lane group, approach, and intersection. The tenth step can be optionally used to estimate lane group queue length and storage ratio.

For actuated control, the methodology is shown to be iterative within Steps 3 to 6, with convergence achieved when the predicted phase duration and capacity from successive iterations are effectively in agreement. Before the first iteration, an initial rough estimate of the phase duration is made to support the calculations in Steps 3 to 5. A revised estimate of phase duration is produced in Step 6. The revised estimate is compared with the previous estimate and, if they are not in agreement, the process is repeated until convergence is achieved. Several iterations are typically needed.

Although not shown in Exhibit 19-18, Step 3 includes an iterative procedure that is used when one or more lane groups have a shared lane. This procedure allocates the through volume among the available shared and exclusive through lanes to determine the lane group volume assignment that produces the lowest service time. It is implemented for both pretimed and actuated control.

Exhibit 19-18
Motorized Vehicle Methodology for Signalized Intersections



COMPUTATIONAL STEPS

Step 1: Determine Movement Groups and Lane Groups

The movement groups and lane groups are established during this step. They are established separately for each intersection approach. Rules for establishing these groups are described in the subsequent paragraphs. Exhibit 19-19 shows some common movement groups and lane groups. A discussion of the need for, and difference between, movement groups and lane groups is provided in a previous subsection titled Lane Groups and Movement Groups.

Number of Lanes	Movements by Lanes	Movement Groups (MG)	Lane Groups (LG)
1	Left, through, and right:	MG 1:	LG 1:
2	Exclusive left: Through and right:	MG 1: MG 2:	LG 1: LG 2:
2	Left and through: Through and right:	MG 1:	LG 1: LG 2:
3	Exclusive left: Exclusive left: Through: Through: Through and right:	MG 1: MG 2:	LG 1: LG 2: LG 3:

Exhibit 19-19
Typical Movement Groups and Lane Groups

Determine Movement Groups

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

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

These rules result in the designation of one to three movement groups for each approach. A movement group can include one or more lanes.

Determine Lane Groups

A lane group can include one or more lanes. 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.

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 for the exclusive lanes. Any of the approach flow that is yet to be assigned to a movement group (after 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. The reduced right-turn volume is used to compute capacity and LOS in subsequent steps.

Step 3: Determine Lane Group Flow Rate

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

If there are one or more shared lanes on the approach and two or more lanes, then the lane group flow rate is computed by the procedure described in Section 2 of Chapter 31 in the subsection titled Lane Group Flow Rate on Multiple-Lane Approaches. This procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection, where the lane volume-to-saturation flow ratio is used to estimate relative differences in this time among lanes. This assumption may not always hold for situations in which drivers choose a lane so they are prepositioned for a turn at the downstream intersection. Similarly, it may not hold when an auxiliary through lane is present. In either 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.

Step 4: Determine Adjusted Saturation Flow Rate

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

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

The procedure described in this step applies to lane groups that consist of an exclusive lane (or lanes) operating in a pretimed protected mode and without pedestrian or bicycle interaction. When these conditions do not hold, the supplemental procedures described in Sections 2 and 3 of Chapter 31 should be combined with the procedures in this step to compute the adjusted saturation flow rate.

Equation 19-8 is used to compute the adjusted saturation flow rate per lane for the subject lane group.

$$s = s_o\, f_w\, f_{HVg}\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{RT}\, f_{Lpb}\, f_{Rpb}\, f_{wz}\, f_{ms}\, f_{sp}$$

Equation 19-8

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_{HVg} = adjustment factor for heavy vehicles and 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,

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

f_{wz} = adjustment factor for work zone presence at the intersection,

f_{ms} = adjustment factor for downstream lane blockage, and

f_{sp} = adjustment factor for sustained spillback.

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

Base Saturation Flow Rate

Computations begin with selection of a base saturation flow rate. This base rate represents the expected average flow rate for a through-traffic lane having geometric and traffic conditions that correspond to a value of 1.0 for each adjustment factor. Typically, one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. Default values for this rate are provided in Exhibit 19-11.

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 19-20.

Exhibit 19-20
Lane Width Adjustment Factor

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

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

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

Adjustment for Heavy Vehicles and Grade

The heavy-vehicle and grade adjustment factor f_{HVg} accounts for the combined effect of heavy vehicle and approach grade on saturation flow rate. The heavy-vehicle component of this factor accounts for the additional space occupied by heavy vehicles and for the difference in their operating capabilities compared with passenger cars. The grade component accounts for the effects of approach grade on vehicle performance. An uphill grade has a positive value and a downhill grade has a negative value.

If the grade is negative (i.e., downhill), then the factor is computed with Equation 19-9.

Equation 19-9

$$f_{HVg} = \frac{100 - 0.79\,P_{HV} - 2.07\,P_g}{100}$$

If the grade is not negative (i.e., level or uphill), then the factor is computed with Equation 19-10.

Equation 19-10

$$f_{HVg} = \frac{100 - 0.78\,P_{HV} - 0.31\,P_g^2}{100}$$

where

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

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

This factor applies to heavy vehicle percentages up to 50% and grades ranging from –4.0% to +10.0%. This factor does not address local buses that stop in the intersection area.

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 19-11.

Equation 19-11

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

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 19-11. A minimum value of f_p from this equation is 0.050. Each maneuver (either in or out) is assumed to block traffic in the lane next to the parking maneuver for an average of 18 s.

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

Adjustment for Bus Blockage

The bus-blockage adjustment factor f_{bb} accounts for the impact of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 250 ft of the stop line (upstream or downstream). Values of this factor are computed with Equation 19-12.

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

Equation 19-12

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

This factor should be used only when stopping buses block traffic flow in the subject lane group. A practical upper limit of 250 buses/h should be maintained with Equation 19-12. A minimum value of f_{bb} from this equation is 0.050. The factor used here assumes an average blockage time of 14.4 s during a green indication.

Adjustment for Area Type

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

Use of this factor should be determined on a case-by-case basis. This factor is not limited to designated CBD areas, nor does it need to be used for all CBD areas. Instead, it should be used in areas where the geometric design and the traffic or pedestrian flows, or both, are such that 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 19-13.

Equation 19-13

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

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

If the right-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Section 3 of Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians and bicycles on right-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Left Turns

The left-turn adjustment factor f_{LT} is intended primarily to reflect the effect of left-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 19-14.

Equation 19-14

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

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

If the left-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Section 3 of Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians on left-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Pedestrians and Bicycles

The procedure to determine the left-turn pedestrian–bicycle adjustment factor f_{Lpb} and the right-turn pedestrian–bicycle adjustment factor f_{Rpb} is based on the concept of conflict zone occupancy, which accounts for the conflict between turning vehicles, pedestrians, and bicycles. Relevant conflict zone occupancy takes into account whether the opposing vehicle flow is also in conflict with the left-turn movement. The proportion of green time in which the conflict zone is occupied is determined as a function of the relevant occupancy and the number of receiving lanes for the turning vehicles. A procedure for computing these factors is provided in Section 2 of Chapter 31. These factors have a value of 1.0 if no pedestrians or bicycles are present.

Adjustment for Work Zone Presence

The adjustment factor for work zone presence f_{wz} is used to evaluate the effect of work zone presence on saturation flow rate. This factor addresses the case in which the work zone is located on the intersection approach. The work zone is considered to be on the intersection approach if some (or all) of the work zone is located between the stop line and a point 250 ft upstream of the stop line. A procedure for computing this factor is provided in Section 2 of Chapter 31, Signalized Intersections: Supplemental. The factor has a value of 1.0 if no work zone is present.

Adjustment for Downstream Lane Blockage

The adjustment factor for downstream lane blockage f_{ms} is used to evaluate the effect of a downstream lane closure on saturation flow rate. A downstream lane closure is a closure located downstream of the subject intersection. The factor is applied only to those lane groups entering the segment on which the closure is present. The lane closure can be associated with a work zone or special event. A procedure for computing this factor is provided in Section 3 of Chapter 30, Urban Street Segments: Supplemental. The factor has a value of 1.0 if no downstream lane blockage is present.

Adjustment for Sustained Spillback

The adjustment factor for sustained spillback f_{sp} is used to evaluate the effect of spillback from the downstream intersection. When spillback occurs, its effect is quantified as a reduction in the saturation flow rate of upstream lane groups entering the segment. A procedure is described in Section 3 of Chapter 29, Urban Street Facilities: Supplemental, for evaluating urban street facilities that experience spillback on one or more segments during the analysis period. The calculation of the adjustment factor for spillback is one part of this procedure. The factor has a value of 1.0 if no spillback occurs.

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 19-15 is used to compute this proportion for each lane group.

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

Equation 19-15

where all variables are as previously defined.

Equation 19-15 requires knowledge of the effective green time g and cycle length C. These values are known for pretimed operation. If the intersection is not pretimed, then the average phase time and cycle length must be calculated by the procedures described in the next step.

A procedure is described in Section 3 of Chapter 30 that can be used to estimate the arrival flow profile for an intersection approach when this approach is evaluated as part of an urban street segment. The procedure uses the profile to compute the proportion of arrivals during the green indication.

Step 6: Determine Signal Phase Duration

The duration of a signal phase depends on the type of control used at the subject intersection. If the intersection has pretimed control, then the phase duration is an input, and the evaluation continues with Step 7. If the phase duration is unknown, then the pretimed phase duration procedure in Section 2 of Chapter 31 can be used to estimate the pretimed phase duration.

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

Step 7: Determine Capacity and 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 19-16.

Equation 19-16

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

where c is the capacity (veh/h), and all other variables are as previously defined. Equation 19-16 cannot be used to calculate the capacity of a shared-lane lane group or a lane group with permitted operation because these lane groups have other factors that affect their capacity. Chapter 31 provides a procedure for estimating the capacity of these types of lane groups.

The volume-to-capacity ratio for a lane group is defined as the ratio of the lane group volume and its capacity. It is computed with Equation 19-17.

Equation 19-17

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

where

X = volume-to-capacity ratio,

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

c = capacity (veh/h).

The critical intersection volume-to-capacity ratio is also computed during this step. Guidelines for computing this ratio are provided in Section 4, Extensions to the Motorized Vehicle Methodology.

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 with Equation 19-18.

Equation 19-18

$$d = d_1 + d_2 + d_3$$

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).

Chapter 31 describes a technique for measuring control delay in the field.

A. Compute Uniform Delay

The uniform delay for a given lane group serving one traffic movement, and for which there are no permitted movements, is computed by using Equation 19-19 with Equation 19-20 and Equation 19-21.

Equation 19-19

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

with

Equation 19-20

$$PF = \frac{1 - P}{1 - g/C} \times \frac{1 - y}{1 - \min(1, X)\; P} \times \left[1 + y \frac{1 - P\; C/g}{1 - g/C}\right]$$

Equation 19-21

$$y = \min(1, X)\; g/C$$

where

- PF = progression adjustment factor,
- y = flow ratio,
- P = proportion of vehicles arriving during the green indication (decimal),
- g = effective green time (s), and
- C = cycle length (s).

Equation 19-19 does not provide an accurate estimate of uniform delay for a shared-lane lane group or a lane group with permitted operation because these lane groups have other factors that affect their delay. Also, this equation does not provide an accurate estimate of uniform delay when there is an initial queue present for one or more intersection traffic movements. Section 4, Extensions to the Motorized Vehicle Methodology, describes a procedure for accurately estimating uniform delay when any of these conditions is present.

B. Compute Initial Queue Delay

The initial queue delay term accounts for the additional uniform delay incurred due to an initial queue. This queue is a result of unmet demand in the previous time period. It does *not* include delay to any vehicles that may be in queue due to the random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. The initial queue delay equals 0.0 s/veh when there is no initial queue present at the start of the analysis period for any intersection lane group. A procedure for estimating the associated delay for lane groups with an initial queue present is provided in Section 4, Extensions to the Motorized Vehicle Methodology.

C. Compute Incremental Delay Factor

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

An actuated phase has the ability to adapt its green interval duration to serve the demand on a cycle-by-cycle basis and, thereby, to minimize the frequency of cycle failure. Only when the green is extended to its maximum limit is this capability curtailed. This influence of actuated operation on delay is accounted for in Equation 19-22 through Equation 19-25.

Equation 19-22

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

with

Equation 19-23

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

Equation 19-24

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

Equation 19-25

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

where

- k = incremental delay factor,
- c_a = available capacity for a lane group served by an actuated phase (veh/h),
- k_{min} = minimum incremental delay factor,
- PT = passage time setting (s),
- G_{max} = maximum green setting (s), and
- g_a = available effective green time (s).

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

D. Compute Incremental Delay

Incremental delay consists of two delay components. One component accounts for delay due to the effect of random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. This delay is evidenced by the occasional overflow queue at the end of the green interval (i.e., cycle failure). The second component accounts for delay due to a sustained oversaturation during the analysis period. This delay occurs when aggregate demand during the analysis period exceeds aggregate capacity. It is sometimes referred to as the deterministic delay component and is shown as variable $d_{2,d}$ in Exhibit 19-21.

Exhibit 19-21 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 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.

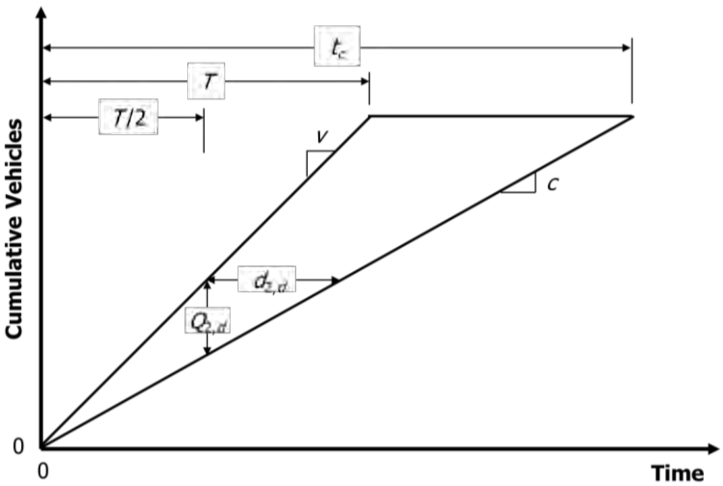


Exhibit 19-21
Cumulative Arrivals and Departures During an Oversaturated Analysis Period

The incremental delay term accounts for delay due to random variation in the number of arrivals on a cycle-by-cycle basis. It also accounts for delay caused by demand exceeding capacity during the analysis period. The amount by which demand exceeds capacity during the analysis period is referred to as unmet demand. The incremental delay equation was derived by using an assumption of no initial queue due to unmet demand in the preceding analysis period. Equation 19-26, with Equation 19-27, is used to compute incremental delay.

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

Equation 19-26

with

$$X_A = v/c_A$$

Equation 19-27

where X_A is the average volume-to-capacity ratio, c_A is the average capacity (veh/h), and all other variables are as previously defined. The variable c_A is not the same as the variable c_a, the latter of which is computed in Part C of Step 8.

If no lane group at the intersection has an initial queue, then the average lane group capacity c_A is equal to the capacity c computed in Step 7 (i.e., $c_A = c$). If one or more lane groups have an initial queue, then the procedure described in Section 4 is used to compute capacity c_A.

The incremental delay term is valid for all values of X_A, including highly oversaturated lane groups.

E. Compute Lane Group Control Delay

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

F. Compute Aggregated Delay Estimates

It is often desirable to compute the average control delay for the intersection approach. This aggregated delay represents a weighted average delay, where each lane group delay is weighted by the lane group demand flow rate. The approach control delay is computed with Equation 19-28.

Equation 19-28

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

where

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

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

m_j = number of lane groups on approach j, and

all other variables are as previously defined. The summation terms in Equation 19-28 represent the sum for all lane groups on the subject approach.

Similarly, intersection control delay is computed with Equation 19-29.

Equation 19-29

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

where d_I is the intersection control delay (s/veh). The summation terms in Equation 19-29 represent the sum for all lane groups at the subject intersection.

Unsignalized movements at the signalized intersection should also be considered when an aggregated delay estimate is computed. Inclusion of these movements should be handled as follows:

- Delay of unsignalized movements should be included in the approach and intersection aggregate delay calculations of Equation 19-28 and Equation 19-29, except for special cases that are properly annotated in the results.
- When the delay of unsignalized movements is included in the approach and intersection averages, whether zero or nonzero, the aggregate delay that results must be annotated with a footnote that indicates this unsignalized delay inclusion.
- When the delay of unsignalized movements is not included in the aggregate totals [i.e., it is not included in either the numerator (volume × delay) or the denominator (volume) of Equation 19-28 or Equation 19-29], this exclusion of unsignalized delay must be clearly represented by a footnote that indicates this unsignalized delay exclusion.

Step 9: Determine LOS

Exhibit 19-8 is used to determine the LOS for each lane group, each approach, and the intersection as a whole. LOS is an indication of the acceptability of delay levels to motorists at the intersection. It can also indicate an unacceptable oversaturated operation for individual lane groups.

Step 10: Determine Queue Storage Ratio

A procedure is described in Section 4 of Chapter 31 for estimating the back-of-queue size and the queue storage ratio. The back-of-queue position is the position of the vehicle stopped farthest from the stop line during the cycle as a consequence of the display of a red signal indication. The back-of-queue size depends on the arrival pattern of vehicles and on the number of vehicles that do not clear the intersection during the previous cycle.

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

Interpretation of Results

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

Level of Service

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

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

Volume-to-Capacity Ratio

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

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

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

- Basic changes in intersection geometry (i.e., change in the number or use of lanes),

- Increase in signal cycle length if it is determined to be too short, and
- Changes in signal phase sequence or timing.

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

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

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

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

Volume-to-Capacity Ratio and Delay Combinations

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

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

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

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

4. EXTENSIONS TO THE MOTORIZED VEHICLE METHODOLOGY

CRITICAL INTERSECTION VOLUME-TO-CAPACITY RATIO

Overview

A useful concept for analyzing signalized intersections is the critical intersection volume-to-capacity ratio X_c. This ratio is computed by using Equation 19-30 with Equation 19-31.

Equation 19-30

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

with

Equation 19-31

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

where

X_c = critical intersection volume-to-capacity ratio,

C = cycle length (s),

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

$l_{t,i}$ = 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 occurs 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 19-2.

Equation 19-30 is based on the combined assumption that each critical phase has the same volume-to-capacity ratio and that this ratio is equal to the critical intersection volume-to-capacity ratio. This assumption is valid when the effective green duration for each critical phase i is proportional to $y_{c,i}/\Sigma(y_{c,i})$. When this assumption holds, the volume-to-capacity ratio for each noncritical phase is less than or equal to the critical intersection volume-to-capacity ratio.

Identifying Critical Lane Groups and Critical Flow Ratios

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

Next, the lane group flow ratio is computed for each lane group served by the phase. If a lane group is served only during one 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., $v_i/(N\ s_i)$]. If a lane group is served during multiple phases (e.g., protected-permitted), 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.

If the lane group is served in a permitted manner, then the saturation flow rate s_i used to determine the flow ratio is an average for the permitted green period. For left turns, it is computed with Equation 31-59 in Chapter 31 (with E_{L2} and E_{L1} substituted for $E_{L2,m}$ and $E_{L1,m}$, respectively) and the instructions that follow this equation as they relate to shared or exclusive lane assignment. This equation applies to lane groups served as permitted-only and to lane groups served during the permitted phase of protected-permitted operation. For right turns, Equation 31-61 is used (with E_R substituted for $E_{R,m}$).

If the lane group is served by protected-permitted operation, then its volume v_i must be apportioned to the protected and permitted phases. To accomplish this apportionment, it is appropriate to consider the phase that is displayed first to be fully saturated by turning traffic and to apply any residual flow to the phase that is displayed second. In this manner, the volume assigned to the first phase is the smaller of the phase capacity or the demand volume, and any unassigned volume goes to the second phase.

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

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

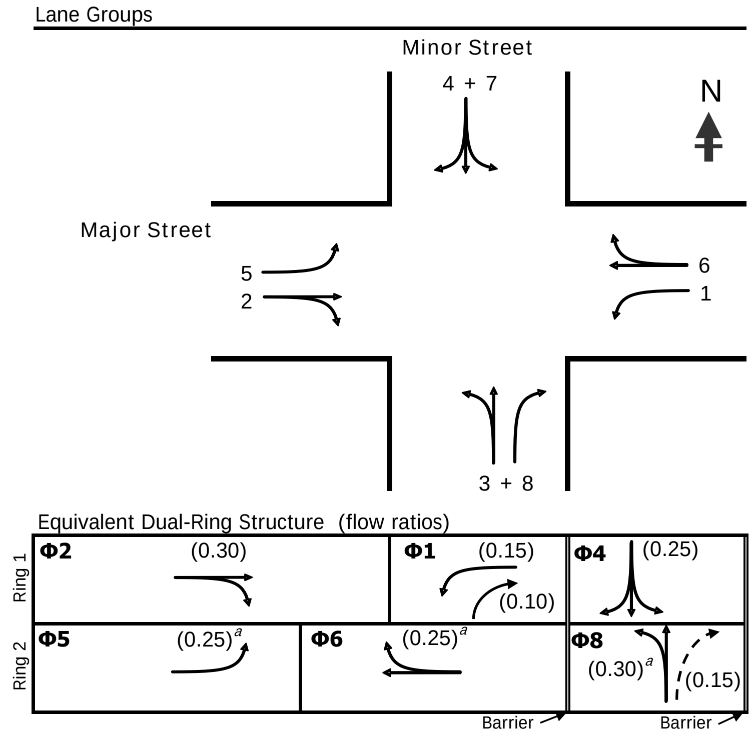
Finally, the process is repeated for the phases between the other barrier pair. One critical flow rate is defined for each barrier pair by this process. These two values are then added to obtain the sum of the critical flow ratios used in Equation 19-30. The lost time associated with each of the critical phases is added to yield the cycle lost time L.

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

Basic Case

For the basic case, consider an intersection with a lead–lag phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 19-22. The northbound right turn is provided an exclusive lane and a green arrow indication that displays concurrently with the complementary left-turn phase on the major street. Each of the left-turn movements on the major street is served with a protected phase.

Exhibit 19-22
Critical Path Determination with Protected Left-Turn Phases



Note: [a] Critical flow ratio.

Phases 4 and 8 represent the only phases between the barrier pair serving the minor-street movements. Inspection of the flow ratios provided in the exhibit indicates 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 larger phase flow ratio is 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. The phase flow ratio of Phase 1 is 0.15, on the basis of the left-turn lane group flow rate.

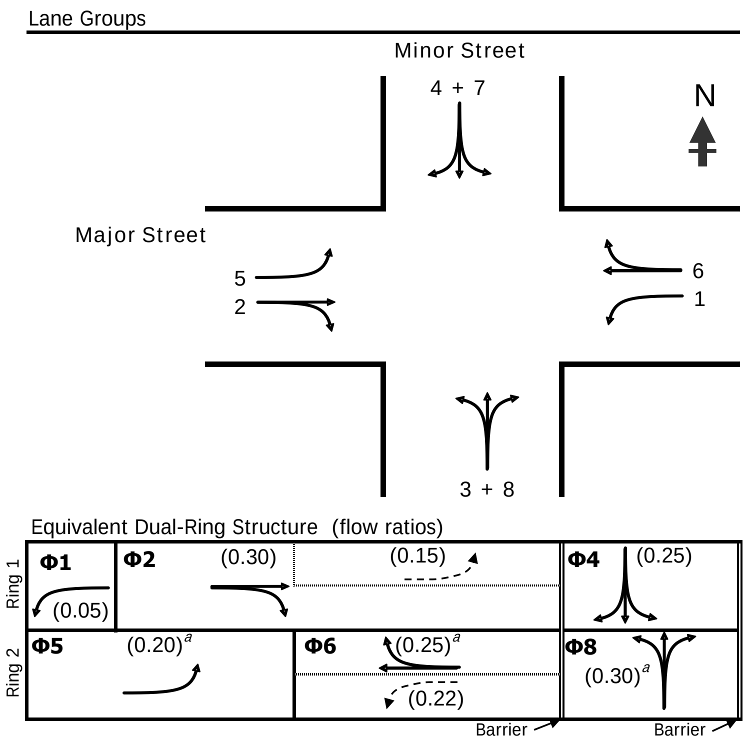
There are two possible critical paths through the major-street phase sequence. One path is associated with Phases 1 and 2 (i.e., Ring 1), and the other path is associated with Phases 5 and 6 (i.e., Ring 2). The total phase flow ratio for the Ring 1 path is 0.30 + 0.15 = 0.45. The total phase flow ratio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

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

Special Case: Protected-Permitted Left-Turn Operation

For the special case, consider an intersection with a lead–lead phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 19-23. 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 19-23
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 19-23. The following rules define the possible critical paths through this phase sequence:

1. One path is associated with Phases 1 and 2 in Ring 1 (0.35 = 0.05 + 0.30).
2. One path is associated with Phases 5 and 6 in Ring 2 (0.45 = 0.20 + 0.25).
3. If a lead–lead or lag–lag phase sequence is used, then one path is associated with (*a*) the left-turn phase with the larger flow ratio and (*b*) the through phase that permissively serves the same left-turn lane group. Sum the protected and permitted left-turn flow ratios on this path (0.35 = 0.20 + 0.15).
4. If a lead–lag phase sequence is used, then one path is associated with (*a*) the leading left-turn phase, (*b*) the lagging left-turn phase, and (*c*) the

controlling through phase (see discussion to follow). Sum the two protected left-turn flow ratios and the one controlling permitted left-turn flow ratio on this path.

If a lead–lag phase sequence is used, each of the through phases that permissively serve a left-turn lane group is considered in determining the controlling through phase. If both through phases have a permitted period, then there are two through phases to consider. The controlling through phase is that phase with the larger permitted left-turn flow ratio. For example, if Phase 1 were shown to lag Phase 2 in Exhibit 19-23, then Phase 6 would be the controlling through phase because the permitted left-turn flow ratio of 0.22 exceeds 0.15. The critical path for this phase sequence would be 0.47 (= 0.20 + 0.22 + 0.05).

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

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

Two flow ratios are associated with Phase 6 in this example. Both flow ratios are shown possibly to dictate the duration of Phase 6 (this condition does not hold for Phase 2 because of the timing of the left-turn phases). This condition is similar to that for the northbound right-turn movement in Phase 1 of Exhibit 19-22, and the treatment is the same. That is, both flow ratios are considered in defining the phase flow ratio for Phase 6.

UNIFORM DELAY CALCULATION USING QUEUE ACCUMULATION POLYGON

Overview

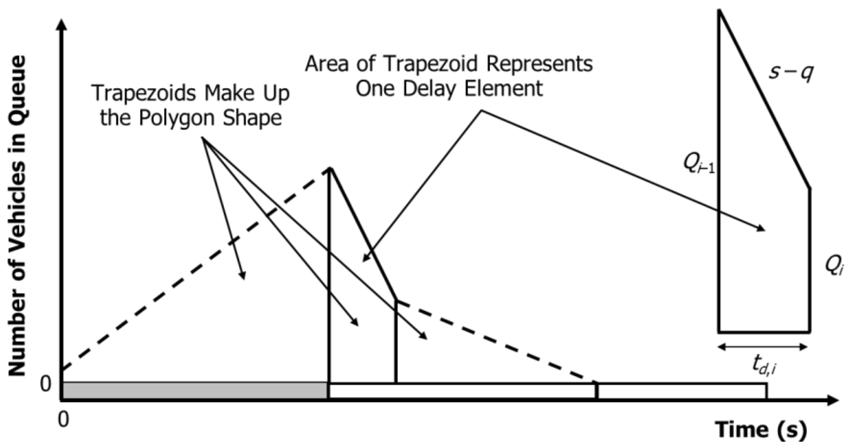
This subsection describes a procedure for calculating uniform delay. This incremental queue accumulation procedure (*21*, *22*) is sufficiently general that it can be applied to any lane group, regardless of whether the lane group is shared or exclusive or served with a protected, permitted, or protected-permitted operation.

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 19-7. The procedure decomposes the resulting polygon into an equivalent set of trapezoids or triangles for the purpose of delay estimation.

Polygon Construction

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 19-24 for a lane group having two different departure rates during the effective green period.

Exhibit 19-24
Decomposition of Queue Accumulation Polygon



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 the polygon is constructed, this assumption must be checked. 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.

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

Polygon construction requires identifying points in the cycle at which 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, departure of sneakers), or
- 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 Data and Sources subsection. The proportion of vehicles arriving during the green indication P is used to compute the arrival flow rate during each flow state. Equation 19-32 and Equation 19-33 can be used to compute these rates.

Equation 19-32

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

and

Equation 19-33

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

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),

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

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

g = effective green time (s).

A more detailed description of the procedure for constructing a queue accumulation polygon for lane groups with various lane allocations and operating modes is provided in Section 3 of Chapter 31.

Delay Calculation

The uniform delay 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 19-34 illustrates this calculation for interval i.

Equation 19-34

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

where

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

N = number of lanes in the lane group (ln),

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

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

The uniform delay is calculated by using Equation 19-35 with Equation 19-36.

$$d_1 = \frac{0.5 \sum (Q_{i-1} + Q_i)\, t_{t,i}}{q\, C}$$

Equation 19-35

with

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

Equation 19-36

where

d_1 = uniform delay (s/veh),

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

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

all other variables are as previously defined.

The summation term in Equation 19-35 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).

INITIAL QUEUE DELAY CALCULATION

Overview

Initial queue delay 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.

Exhibit 19-25 illustrates the delay due to an initial queue as a trapezoid shape bounded by thick lines. The average delay per vehicle is represented by the variable d_3. The initial queue size is shown as Q_b vehicles. The duration of time during the analysis period for which the effect of the initial queue is still present is represented by the variable t. This duration is shown to equal the analysis period T in Exhibit 19-25. However, it can be shorter than the analysis period duration for some lower-volume conditions.

Exhibit 19-25 illustrates the case in which the demand flow rate v exceeds the capacity c during the analysis period. In contrast, Exhibit 19-26 and Exhibit 19-27 illustrate alternative cases in which the demand flow rate is less than the capacity.

The remainder of this subsection describes the procedure for computing the initial queue delay for a lane group during a given analysis period.

Exhibit 19-25
Initial Queue Delay with Increasing Queue Size

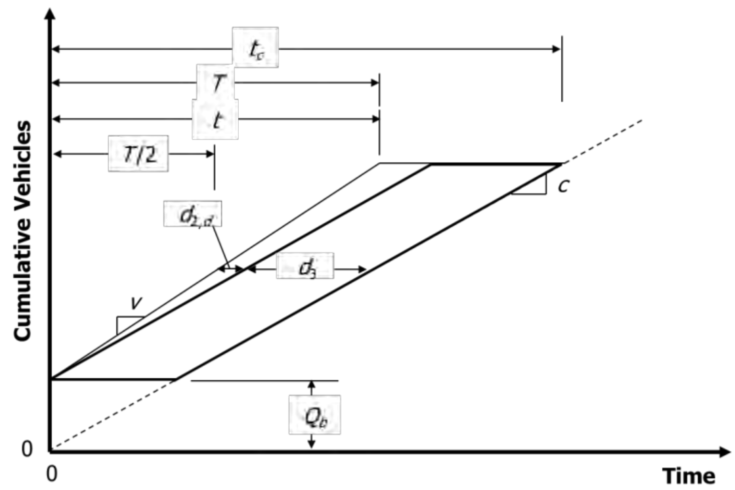


Exhibit 19-26
Initial Queue Delay with Decreasing Queue Size

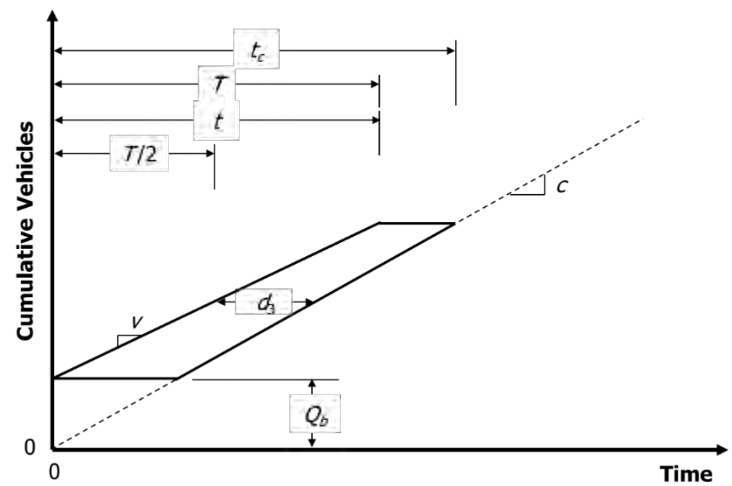
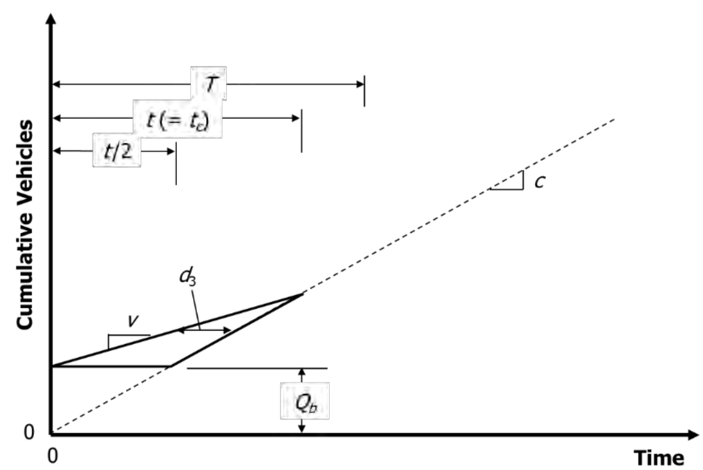


Exhibit 19-27
Initial Queue Delay with Queue Clearing



Computational Steps

A. Initial Queue Analysis

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

The saturation flow rate, phase duration, capacity, and uniform delay will need to be recomputed for each lane group during this step. When these variables are computed for a lane group with an initial queue, the arrival flow rate for the lane group is inflated such that it equals the lane group capacity (i.e., the actual input demand flow rate is not used). The remaining lane groups will have their arrival flow rate set to equal the smaller of the input demand flow rate or the capacity.

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

The uniform delay computed during this step is referred to as the saturated uniform delay. It is computed for each lane group by using the arrival flow rate, capacity, and phase duration determined with the previous guidance.

The duration of unmet demand is calculated in this step for each lane group. Either Equation 19-37 or Equation 19-38 is used for this purpose.

If $v \geq c_s$, then

$$t = T$$

Equation 19-37

If $v < c_s$, then

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

Equation 19-38

where

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

T = analysis period duration (h),

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

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

c_s = saturated capacity (veh/h).

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

Next, the average duration of unmet demand is calculated with Equation 19-39.

Equation 19-39

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

where

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

t_i = duration of unmet demand for lane group i in the analysis period (h), and

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

The summation term in Equation 19-39 represents the sum of the t values for only those lane groups that have a value of t that exceeds 0.0 h. The average duration t_a is considered as a single representative value of t for all lane groups that do not have an initial queue.

B. Compute Uniform Delay

The uniform delay computed in Step 8 of the core motorized vehicle methodology is adjusted in this step such that the adjusted uniform delay reflects the presence of the initial queue. Initially, the uniform delay d_1 computed previously is renamed as the baseline uniform delay d_{1b} (i.e., $d_{1b} = d_1$). Next, Equation 19-40 or Equation 19-41 is used to compute the uniform delay for each lane group.

If lane group i has an initial queue, then

Equation 19-40

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

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

Equation 19-41

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

where d_s is the saturated uniform delay (s/veh), d_{1b} is the baseline uniform delay (s/veh), t_i is the duration of unmet demand for lane group i in the analysis period (h), and other variables are as previously defined.

C. Compute Average Capacity

Equation 19-42 and Equation 19-43 are used to compute the average capacity for each lane group.

If lane group i has an initial queue, then

Equation 19-42

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

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

$$c_{A,i} = c_{s,i}\frac{t_a}{T} + c_i\frac{(T - t_a)}{T} \quad \text{Equation 19-43}$$

where c_A is the average capacity (veh/h).

D. Compute Initial Queue Delay

Equation 19-44 through Equation 19-49 are used to compute the initial queue delay for each lane group.

$$d_3 = \frac{3{,}600}{v\,T}\left(t_A\frac{Q_b + Q_e - Q_{eo}}{2} + \frac{Q_e^2 - Q_{eo}^2}{2\,c_A} - \frac{Q_b^2}{2\,c_A}\right) \quad \text{Equation 19-44}$$

with

$$Q_e = Q_b + t_A(v - c_A) \quad \text{Equation 19-45}$$

If $v \geq c_A$, then

$$Q_{eo} = T\,(v - c_A) \quad \text{Equation 19-46}$$

$$t_A = T \quad \text{Equation 19-47}$$

If $v < c_A$, then

$$Q_{eo} = 0.0 \text{ veh} \quad \text{Equation 19-48}$$

$$t_A = Q_b/(c_A - v) \leq T \quad \text{Equation 19-49}$$

where

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

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

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

all other variables are as previously defined. The queue at the end of the analysis period Q_e is also referred to as the residual queue.

The last vehicle that arrives to an overflow queue during the analysis period will clear the intersection at the time obtained with Equation 19-50.

$$t_c = t_A + Q_e/c_A \quad \text{Equation 19-50}$$

where t_c is the queue-clearing time (h).

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

5. PEDESTRIAN METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to pedestrians traveling through a signalized intersection.

SCOPE OF THE METHODOLOGY

The overall scope of the three methodologies was provided in Section 2. This section identifies the additional conditions for which the pedestrian methodology is applicable.

- *Target travel modes*. The pedestrian methodology addresses travel by pedestrians walking across one or more legs of a signalized intersection. It is not designed to evaluate the performance of other travel means (e.g., Segway, roller skates).
- *"Typical pedestrian" focus for pedestrian methodology*. The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. The performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to Americans with Disabilities Act requirements. For this reason, they should not be considered as a substitute for an Americans with Disabilities Act compliance assessment of a pedestrian facility.

Spatial Limits

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

Performance Measures

Performance measures applicable to the pedestrian travel mode include pedestrian delay, pedestrian LOS score, corner circulation area, and crosswalk circulation area. Pedestrian delay represents the average time a pedestrian waits for a legal opportunity to cross an intersection leg. The LOS score is an indication of the typical pedestrian's perception of the overall crossing experience.

LOS is also considered a performance measure. It is useful for describing intersection performance to elected officials, policy makers, administrators, or the public. LOS is based on the pedestrian LOS score.

The two circulation-area performance measures are based on the concept of pedestrian space. One measure is used to evaluate the circulation area provided to pedestrians while they wait at the corner. The other measure is used to evaluate the area provided while the pedestrian is crossing in the crosswalk.

Limitations of the Methodology

This subsection identifies the known limitations of the pedestrian methodology. If one or more of these limitations is believed to have an important influence on the performance of a specific intersection, then the analyst should consider using alternative methods or tools for the evaluation.

The pedestrian methodology does not account for the effect of the following conditions on the quality of service provided to pedestrians:

- Grades in excess of 2%,
- Presence of railroad crossings,
- Unpaved sidewalk, and
- Free (i.e., uncontrolled) channelized right turn with multiple lanes or high-speed operation.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the pedestrian methodology. These data are listed in Exhibit 19-28. The second column (labeled Basis) of the exhibit indicates whether the input data are needed for each traffic movement, each signal phase, each intersection approach, or the intersection as a whole. The third column [labeled Performance Measure(s)] indicates which performance measures the input is required for. The exhibit also lists default values that can be used if local data are not available (*25*, *30*).

The data elements listed in Exhibit 19-28 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during presentation of the methodology.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 19-28. These data describe the traffic streams traveling through the intersection during the study period. The demand flow rate of motorized vehicles and RTOR flow rate are defined in Section 3 for the motorized vehicle mode.

Permitted Left-Turn Flow Rate

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

Exhibit 19-28
Required Input Data, Potential Data Sources, and Default Values for Pedestrian Analysis

Required Data and Units	Basis	Performance Measure(s)	Potential Data Source(s)	Suggested Default Value
Traffic Characteristics				
Motorized vehicle demand flow rate (veh/h)	M	LOS, XWCA	Field data, past counts	Must be provided
Right-turn-on-red flow rate (veh/h)	A	LOS	Field data, past counts	Must be provided
Permitted left-turn flow rate (veh/h)	M	LOS	Field data, past counts	See discussion
Midsegment 85th percentile speed (mi/h)	A	LOS	Field data	Speed limit
Pedestrian flow rate (veh/h)	M	XWCA, CCA	Field data, past counts	Must be provided
Geometric Design				
Number of lanes (ln)	L	Delay, LOS	Field data, aerial photo	Must be provided
Number of right-turn islands (0, 1, 2)	L	LOS	Field data, aerial photo	0
Total walkway width (ft)	A	CCA	Field data, aerial photo	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
Crosswalk width (ft)	L	XWCA	Field data, aerial photo	12 ft
Crosswalk length (ft)	L	2-stage delay, XWCA	Field data, aerial photo	Must be provided
Corner radius (ft)	A	CCA	Field data, aerial photo	Trucks and buses in turn volume: 45 ft No trucks or buses in turn volume: 25 ft
Signal Control				
Walk (s)	P	Delay, LOS	Field data	Actuated: 7 s Pretimed: green interval minus pedestrian clear
Pedestrian clear (s)	P	Delay, LOS	Field data	Based on 3.5-ft/s walking speed
Rest in walk (yes or no)	P	Delay, LOS	Field data	Not enabled
Cycle length (s)	I	Delay, LOS	Field data	Same as motorized vehicle mode
Yellow change + red clearance (s)[a]	P	Delay, LOS	Field data	4 s
Duration of phase serving pedestrians (s)	P	Delay, LOS	Field data	Same as motorized vehicle mode
Pedestrian signal head presence (yes or no)	P	Delay, LOS	Field data	Must be provided
Other Data				
Analysis period duration (h)[b]	I	Delay, LOS, XWCA, CCA	Set by analyst	0.25 h

Notes: M= movement: one value for each left-turn, through, and right-turn movement.
A = approach: one value for the intersection approach.
L = leg: one value for the intersection leg (approach plus departure sides).
P = phase: one value or condition for each signal phase.
I = intersection: one value for the intersection.
XWCA = crosswalk circulation area, CCA = corner circulation area.
[a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.
[b] Analysis period duration is as defined for Exhibit 19-11.

Default Value. 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. However, a default flow rate can be used if the analysis involves future conditions or if the permitted left-turn flow rate is not known from field data.

The default permitted left-turn flow rate for movements served by the permitted mode is equal to the left-turn demand flow rate.

The default permitted left-turn flow rate for movements served by the protected-permitted mode is equal to the left-turn arrival rate during the permitted period. This arrival rate is estimated as the left-turn flow rate during the effective red time [i.e., $q_r = (1 - P)\ q\ C/r$].

Midsegment 85th Percentile Speed

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

Pedestrian Flow Rate

The pedestrian flow rate represents the count of pedestrians traveling through each corner of the intersection divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. This flow rate is provided for each of five movements at each intersection corner. These five movements (i.e., v_{ci}, v_{co}, v_{di}, v_{do}, and $v_{a,b}$) are shown in Exhibit 19-29 as they occur at one intersection corner.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 19-28. These data describe the geometric elements that influence intersection performance from a pedestrian perspective. The number-of-lanes variable is defined in Section 3 for the motorized vehicle mode.

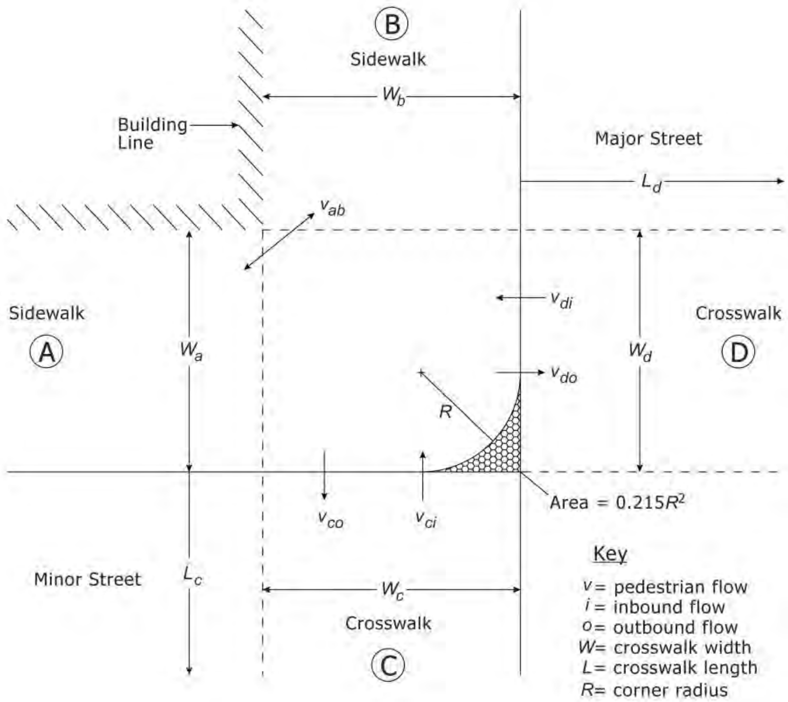
Number of Right-Turn Islands

The number of right-turn islands represents the count of channelizing islands encountered by pedestrians while crossing one intersection leg. The island should be delineated by a raised curb and of sufficient size to be considered a refuge for pedestrians. The number provided must have a value of 0, 1, or 2.

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

The geometric design data of total walkway width, crosswalk width and length, and corner radius describe the pedestrian accommodations on each corner of the intersection. These data are shown in Exhibit 19-29. The total walkway width (i.e., W_a and W_b) is measured from the outside edge of the road pavement (or face of curb, if present) to the far edge of the sidewalk (as sometimes delineated by a building face, fence, or landscaping).

Exhibit 19-29
Intersection Corner Geometry and Pedestrian Movements



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

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

Signal Control Data

This subsection describes the data in Exhibit 19-28 that are identified as signal control. The walk, pedestrian clear, yellow change, and red clearance settings are defined in Section 3 for the motorized vehicle mode.

Rest in Walk

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

Cycle Length

Cycle length is predetermined for pretimed or coordinated-actuated control. Chapter 31 provides a procedure for estimating a reasonable cycle length for these two types of control when cycle length is unknown. Default values for cycle length are defined in Section 3 of the present chapter for the motorized vehicle mode.

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

Duration of Phase Serving Pedestrians

The duration of each phase that serves a pedestrian movement is a required input. This phase is typically the phase that serves the through movement that is adjacent to the sidewalk and for which the pedestrian, bicycle, and through vehicle travel paths are parallel. For example, Phases 2, 4, 6, and 8 are the phases serving the pedestrian movements in Exhibit 19-2.

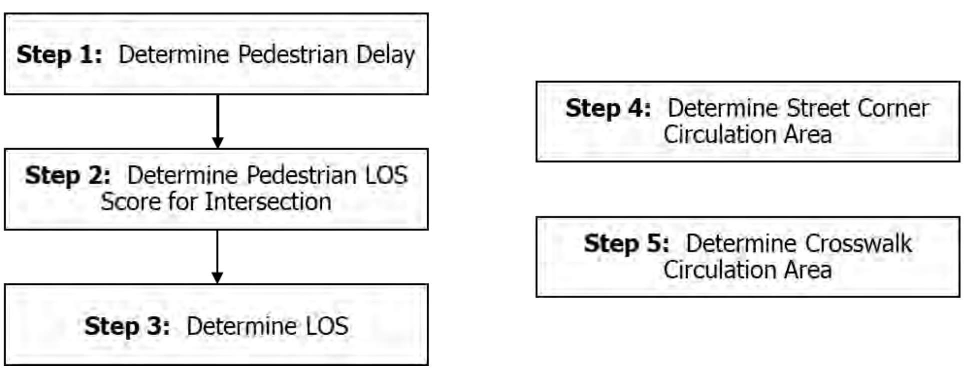
Pedestrian Signal Head Presence

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

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to pedestrians. The methodology is applied through a series of three steps that determine the pedestrian LOS for a signalized pedestrian crosswalk. Two optional additional steps evaluate the operation of a crosswalk, its associated corners, or both. These steps are illustrated in Exhibit 19-30.

Exhibit 19-30
Pedestrian Methodology for Signalized Intersections



The methodology is focused on the analysis of signalized intersection performance. Chapter 18, Urban Street Segments, and Chapter 20, Two-Way STOP-Controlled Intersections, describe methodologies for evaluating the performance of these system elements with respect to the pedestrian mode.

COMPUTATIONAL STEPS

Step 1: Determine Pedestrian Delay

This step describes a procedure for evaluating the average pedestrian delay for crossing one intersection leg or midblock crosswalk in one stage. The Extensions to the Pedestrian Methodology subsection at the end of Section 5. provides procedures for the following additional situations:

- Crossing one intersection leg in two stages (phases), and
- Crossing two intersection legs in two stages.

The procedure is repeated for each crosswalk of interest.

Determining the Effective Walk Time

Research indicates pedestrians typically continue to enter intersections with pedestrian signal heads during the first few seconds of the pedestrian clear interval (*26*, *31*). This behavior effectively increases the effective walk time. A conservative estimate of this additional walk time is 4.0 s (*26*). A nonzero value for this additional time implies some pedestrians are initiating their crossing during the flashing DON'T WALK indication.

The following guidance is provided to estimate the effective walk time on the basis of these research findings (*26*, *32*). If the phase providing service to the pedestrians is either (*a*) actuated with a pedestrian signal head and rest in walk is *not* enabled or (*b*) pretimed with a pedestrian signal head, then Equation 19-51 is used.

Equation 19-51

$$g_{\text{Walk},i} = \text{Walk}_i + 4.0$$

If the phase providing service to the pedestrians is actuated with a pedestrian signal head and rest in walk is enabled, then Equation 19-52 is used.

Equation 19-52

$$g_{\text{Walk},i} = D_{p,i} - Y_i - R_{c,i} - PC_i + 4.0$$

If otherwise (i.e., there is no pedestrian signal head), Equation 19-53 is used.

Equation 19-53

$$g_{\text{Walk},i} = D_{p,i} - Y_i - R_{c,i}$$

where

$g_{\text{Walk},i}$ = effective walk time for the phase serving pedestrian movement *i* (s),

Walk_i = pedestrian walk setting for the phase serving pedestrian movement *i* (s),

PC_i = pedestrian clear setting for the phase serving pedestrian movement *i* (s),

$D_{p,i}$ = duration of the phase serving pedestrian movement *i* (s),

Y_i = yellow change interval of the phase serving pedestrian movement *i* (s), and

$R_{c,i}$ = red clearance interval of the phase serving pedestrian movement *i* (s).

The effective walk time can vary widely among intersections (*26*, *32*). At a given intersection, the additional walk time can vary from 0.0 s to an amount equal to the pedestrian clear interval. The amount of additional walk time used by pedestrians depends on many factors, including the extent of pedestrian delay, vehicular volume, level of enforcement, and the presence of countdown pedestrian signal heads.

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

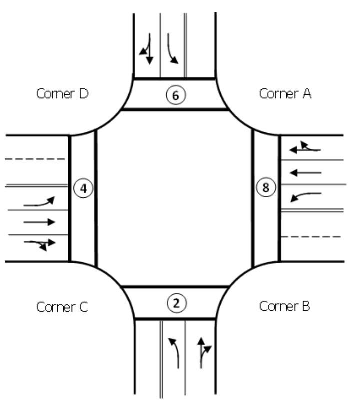
Estimating Delay

The following procedure estimates delay for pre-timed or coordinated semi-actuated signal operation with no permissive period provided for the pedestrian phase. The Extensions to the Pedestrian Methodology subsection at the end of Section 5. provides procedures for the following additional situations:

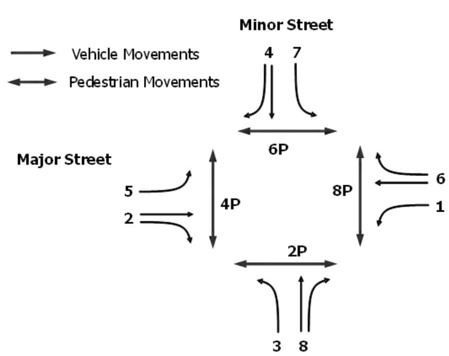
- Coordinated signal operation with a permissive period provided for the pedestrian phase,
- Free signal operation,
- Non-random arrivals, and
- Pedestrian progression through a series of signals.

Exhibit 19-31(a) indicates the number assigned to each crosswalk for a one-stage crossing. Exhibit 19-31(b) indicates the number assigned to each intersection traffic movement. The numbers shown in Exhibit 19-31(b) are established to be coincident with the signal phase that serves the corresponding traffic movement. Notably, the pedestrian movement and the adjacent through vehicle movement share the same number because they are served during the same phase. For example, vehicle movement 2 is a through movement on the left side of the intersection. This movement is served by signal phase 2. Pedestrian movement 2P crosses in crosswalk number 2. This pedestrian movement is also served by signal phase 2.

Exhibit 19-31
Crosswalk, Corner, and Movement Numbering Scheme for One-Stage Crossings



(a) Crosswalk numbering scheme



(b) Traffic movement numbering scheme

The pedestrian delay is computed by Equation 19-54.

Equation 19-54

$$d_{p,i} = \frac{\left(C - g_{\mathrm{Walk},i}\right)^2}{2\,C}$$

where

$d_{p,i}$ = pedestrian delay for pedestrian movement i (s/p).

$g_{\mathrm{Walk},i}$ = effective walk time for the phase serving pedestrian movement i (s), and

C = cycle length (s).

This equation is based on the following three assumptions: (*a*) pedestrian arrivals to the crossing location (i.e., street corner) are random, (*b*) the signal operation is such that pedestrians can cross the entire width of the intersection leg (corner-to-corner) during one signal phase, and (*c*) random arrivals over an large number of signal cycles can be modeled deterministically using a uniform arrival rate. See the Extensions to the Pedestrian Methodology subsection at the end of Section 5. for guidance on other situations. The delay obtained from Equation 19-54 applies equally to both directions of travel along the crosswalk.

Interpreting Delay Results

Research indicates average pedestrian delay at signalized intersection crossings is not constrained by capacity, even when pedestrian flow rates reach 5,000 p/h (*26*). For this reason, delay due to oversaturated conditions is not included in the value obtained from Equation 19-54.

The pedestrian delay computed in this step can be used to make some judgment about pedestrian compliance. In general, pedestrians become impatient when they experience delays in excess of 30 s/p, and there is a high likelihood of their not complying with the signal indication (*33*). In contrast, pedestrians are very likely to comply with the signal indication if their expected delay is less than 10 s/p.

Step 2: Determine Pedestrian LOS Score for Intersection

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

The procedure that follows describes the evaluation of Crosswalk D in Exhibit 19-32. The procedure is repeated to evaluate Crosswalk C in Exhibit 19-32. For the second application, the subscript letter d is replaced with the letter c to denote the length and width of Crosswalk C. Also, the subscript letters mj are replaced with mi to denote variables associated with the minor street.

The pedestrian LOS score for the intersection $I_{p,int}$ is calculated by using Equation 19-55 through Equation 19-60.

Equation 19-55

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

with

$$F_w = 0.681(N_d)^{0.514}$$ Equation 19-56

$$F_v = 0.00569\left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,d}(0.0027\, n_{15,mj} - 0.1946)$$ Equation 19-57

$$F_S = 0.00013\, n_{15,mj}\, S_{85,mj}$$ Equation 19-58

$$F_{\text{delay}} = 0.0401\, ln\, (d_{p,d})$$ Equation 19-59

$$n_{15,mj} = \frac{0.25}{N_d} \sum_{i \in m_d} v_i$$ Equation 19-60

where

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

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_{delay} = pedestrian delay adjustment factor,

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

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

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

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

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

m_d = set of all motorized vehicle movements that cross Crosswalk D (see figure in margin),

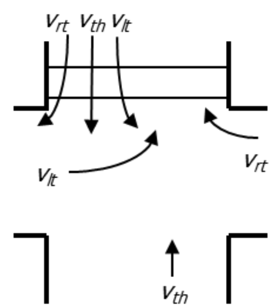
$S_{85,mj}$ = 85th percentile speed at a midsegment location on the major street (mi/h), and

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

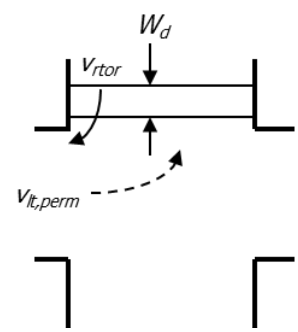
The left-turn flow rate $v_{lt,perm}$ used in Equation 19-57 is the flow rate associated with the left-turn movement that receives a green indication concurrently with the subject pedestrian crossing *and* turns across the subject crosswalk. The RTOR flow rate v_{rtor} is the flow rate associated with the approach being crossed and that also turns across the subject crosswalk. It is not the same v_{rtor} used in Equation 19-70.

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

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



Movements crossing Crosswalk D



Step 3: Determine LOS

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

The pedestrian LOS is determined by using the pedestrian LOS score from Step 4. This performance measure is compared with the thresholds in Exhibit 19-9 to determine the LOS for the subject crosswalk.

Step 4: Determine Street Corner Circulation Area

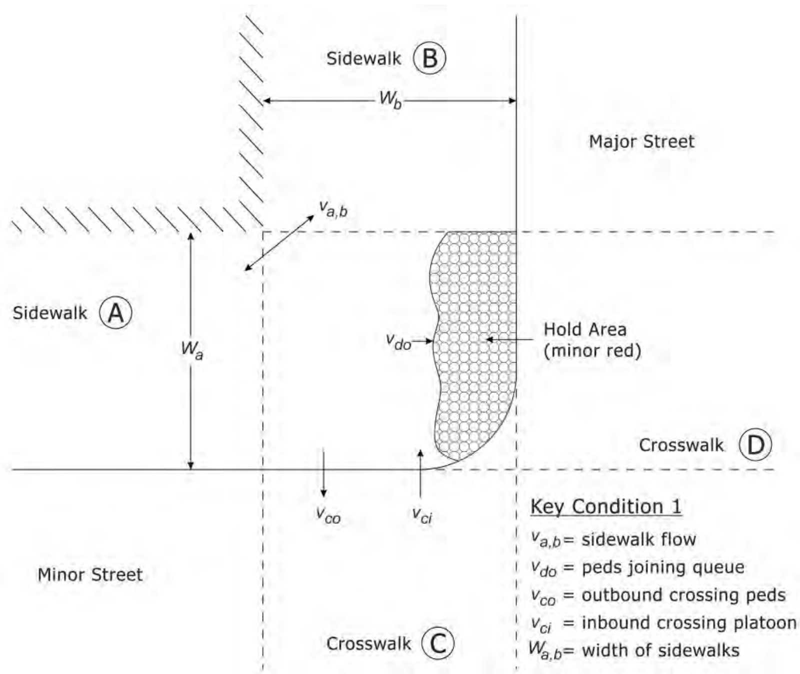
This optional step describes a procedure for evaluating the performance of one intersection corner. It is repeated for each intersection corner of interest. This step can be performed to identify the required area to accommodate all of the following: (*a*) pedestrians waiting to cross one or both crosswalks, (*b*) pedestrians moving across one of the crosswalks, and (*c*) pedestrians turning the corner and not using a crosswalk.

The analysis of circulation area at the street corners and in the crosswalks (Step 5) compares available time and space with pedestrian demand. The product of time and space is the critical parameter. It combines the constraints of physical design (which limits available space) and signal operation (which limits available time). This parameter is referred to as time–space.

Pedestrian Flow Conditions

Exhibit 19-32 and Exhibit 19-33 show the variables considered when one corner and its two crosswalks are evaluated. Two flow conditions are illustrated. Condition 1 corresponds to the minor-street crossing that occurs during the major-street through phase. The pedestrians who desire to cross the major street must wait at the corner. Condition 2 corresponds to the major-street crossing that occurs during the minor-street through phase. For this condition, the pedestrians who desire to cross the minor street wait at the corner.

Exhibit 19-32
Condition 1: Minor-Street Crossing



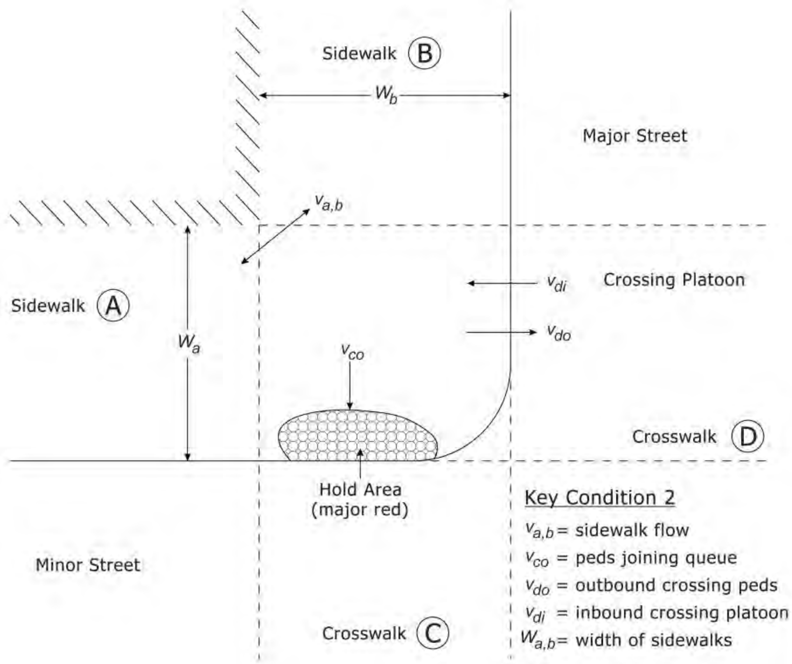


Exhibit 19-33
Condition 2: Major-Street Crossing

Step 4a. Compute Available Time–Space

The total time–space available for circulation and queuing in the intersection corner equals the product of the net corner area and the cycle length C. Equation 19-61 is used to compute the time–space available at an intersection corner. Exhibit 19-29 identifies the variables used in the equation.

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

Equation 19-61

where

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

C = cycle length (s),

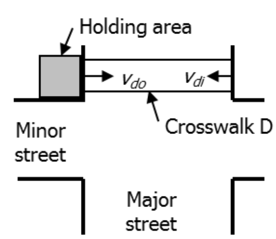
W_a = total walkway width of Sidewalk A (ft),

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

R = radius of corner curb (ft).

If the corner curb radius is larger than either W_a or W_b, then the variable R in Equation 19-61 should equal the smaller of W_a or W_b.

Step 4b. Compute Holding-Area Waiting Time



The average pedestrian holding time represents the average time that pedestrians wait to cross the street when departing from the subject corner. The equation for computing this time is based on the assumption that pedestrian arrivals are uniformly distributed during the cycle.

Condition 1: Minor-Street Crossing

For Condition 1 (shown in Exhibit 19-32), Equation 19-62, with Equation 19-63, is used to compute holding-area time for pedestrians waiting to cross the major street.

Equation 19-62

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

with

Equation 19-63

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

where

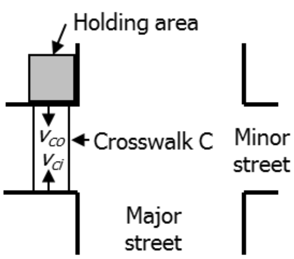
Q_{tdo} = total time spent by pedestrians waiting to cross the major street during one cycle (p-s),

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

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

C = cycle length (s), and

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



Condition 2: Major-Street Crossing

For Condition 2, as shown in Exhibit 19-33, the previous equations are repeated to compute the holding-area time for pedestrians waiting to cross the minor street Q_{tco}. For this application, the subscript letters *do* are replaced with the letters *co* to denote the pedestrians arriving at the corner to cross in Crosswalk C. Similarly, the subscript letters *mi* are replaced with *mj* to denote signal-timing variables associated with the phase serving the major-street through movement.

Step 4c. Compute Circulation Time–Space

The time–space available for circulating pedestrians equals the total available time–space minus the time–space occupied by the pedestrians waiting to cross. The latter value equals the product of the total waiting time and the area used by waiting pedestrians (= 5.0 ft²/p). Equation 19-64 is used to compute the time–space available for circulating pedestrians.

Equation 19-64

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

where TS_c is the time–space available for circulating pedestrians (ft²-s).

Step 4d. Compute Pedestrian Corner Circulation Area

The space required for circulating pedestrians is computed by dividing the time–space available for circulating pedestrians by the time pedestrians consume walking through the corner area. The latter quantity equals the total circulation volume multiplied by the assumed average circulation time (= 4.0 s). Equation 19-65, with Equation 19-66, is used to compute corner circulation area.

Equation 19-65

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

with

Equation 19-66

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

where

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

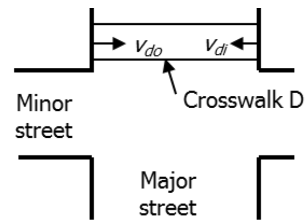
N_{tot} = total number of circulating pedestrians who arrive each cycle (p),

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

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

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

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



all other variables are as previously defined.

The circulation area obtained from Equation 19-65 can be compared with the ranges provided in Exhibit 19-34 to make some judgments about the performance of the subject intersection corner.

Circulation area describes the space available to the average pedestrian. A larger area is more desirable from the pedestrian perspective. Exhibit 19-34 can be used to evaluate intersection circulation area performance from the pedestrian perspective.

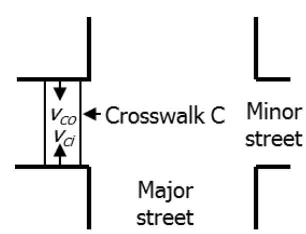
Exhibit 19-34
Qualitative Description of Pedestrian Space

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

Step 5: Determine Crosswalk Circulation Area

This optional step describes a procedure for evaluating the performance of one crosswalk. It is repeated for each crosswalk of interest. The procedure can be used to determine the crosswalk width required to provide a desired crosswalk performance.

The procedure that follows describes the evaluation of Crosswalk D in Exhibit 19-33 (i.e., a crosswalk across the major street). The procedure is repeated to evaluate Crosswalk C in Exhibit 19-32. For the second application, the subscript letters *do* and *di* are replaced with the letters *co* and *ci*, respectively, to denote the pedestrians associated with Crosswalk C. Similarly, the subscript letter *d* is replaced with the letter *c* to denote the length and width of Crosswalk C. Also, the subscript letters *mi* are replaced with *mj* to denote signal-timing variables associated with the phase serving the major-street through movement.



Step 5a. Establish Walking Speed

The average pedestrian walking speed S_p is needed to evaluate corner and crosswalk performance. In the absence of local data, an average pedestrian walking speed of 4.0 ft/s is recommended when less than 20 percent of the pedestrians are elderly (i.e., age 65 or older), and 3.3 ft/s otherwise (*26*).

Step 5b. Compute Available Time–Space

Equation 19-67 is used to compute the time–space available in the crosswalk.

Equation 19-67

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

where

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

L_d = length of Crosswalk D (ft),

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

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

Step 5c. Compute Effective Available Time–Space

The available crosswalk time–space is adjusted in this step to account for the effect turning vehicles have on pedestrians. This adjustment is based on the assumed occupancy of a vehicle in the crosswalk. The vehicle occupancy is computed as the product of vehicle swept-path, crosswalk width, and the time the vehicle preempts this space. Equation 19-68 through Equation 19-70 are used for this purpose.

Equation 19-68

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

with

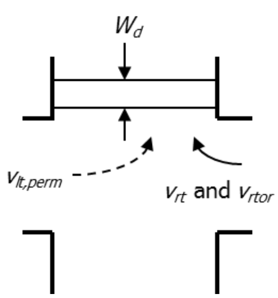
Equation 19-69

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

Equation 19-70

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

where



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

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

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

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

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

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

The constant "40" in Equation 19-69 represents the product of the swept-path for most vehicles (= 8 ft) and the time a turning vehicle occupies the crosswalk (= 5 s). The left-turn and right-turn flow rates used in Equation 19-70 are those associated with movements that receive a green indication concurrently with the subject pedestrian crossing *and* turn across the subject crosswalk.

Step 5d. Compute Pedestrian Service Time

Total service time is computed with either Equation 19-71 or Equation 19-72, depending on the crosswalk width, along with Equation 19-73. This time represents the elapsed time starting with the first pedestrian's departure from the corner to the last pedestrian's arrival at the far side of the crosswalk. In this manner, it accounts for platoon size in the service time (*34*).

If crosswalk width W_d is greater than 10 ft, then

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

Equation 19-71

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

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

Equation 19-72

with

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

Equation 19-73

where

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

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

Equation 19-73 estimates the number of pedestrians who cross as a group following the presentation of the WALK indication (or green indication, if pedestrian signal heads are not provided). It is also used to compute $N_{ped,di}$ for the other travel direction in the same crosswalk (using N_{di}, as defined below). Equation 19-71 or Equation 19-72 is used to compute the service time for pedestrians who arrive at the subject corner having waited on the other corner before crossing the major street $t_{ps,di}$ (using $N_{ped,di}$).

Step 5e. Compute Crosswalk Occupancy Time

The total crosswalk occupancy time is computed as a product of the pedestrian service time and the number of pedestrians using the crosswalk during one signal cycle. Equation 19-74 is used, with Equation 19-75 and results from previous steps, for the computation.

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

Equation 19-74

with

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

Equation 19-75

where

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

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

Step 5f. Compute Pedestrian Crosswalk Circulation Area

The circulation space provided for each pedestrian is determined by dividing the time–space available for crossing by the total occupancy time, as shown in Equation 19-76.

Equation 19-76

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

where M_{cw} is the crosswalk circulation area per pedestrian (ft²/p).

The circulation area obtained from Equation 19-76 can be compared with the ranges provided in Exhibit 19-34 to make some judgments about the performance of the subject-intersection crosswalk (for the specified direction of travel). For a complete picture of the subject crosswalk's performance, the procedure described in this step should be repeated for the other direction of travel along the crosswalk (i.e., by using the other corner associated with the crosswalk as the point of reference).

EXTENSIONS TO THE PEDESTRIAN DELAY METHODOLOGY

This subsection provides additional procedures that can be applied when the assumptions of the core pedestrian delay methodology, described in Step 1 above, do not hold.

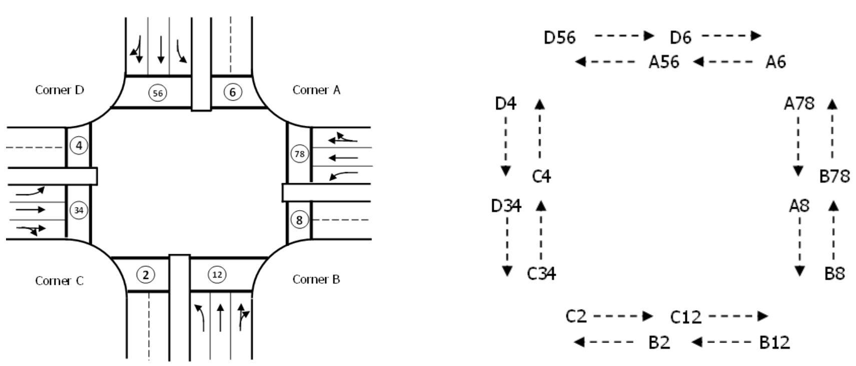
Crossing One Intersection Leg in Two Stages

This procedure computes the average delay to pedestrians that cross a specified intersection leg in a specified direction in two signal phases (*35*). The procedure can be applied again to evaluate the other direction of travel in the specified crosswalk or to evaluate other crosswalks at the intersection.

This procedure is based on the following two assumptions: (*a*) pedestrian arrivals to the first crossing location (i.e., street corner) are random and (*b*) the signal operation is such that pedestrians will need two phases to complete the crossing (waiting on the median before crossing the second half of the street).

The procedure uses the vehicle movement numbering scheme shown in Exhibit 19-31(b). These vehicle movement numbers correspond to the signal phase that serves the movement (i.e., vehicle movement 2 is served by signal phase 2), which follows the traditional eight-phase dual-ring structure. The procedure is also based on the crosswalk numbering scheme shown in Exhibit 19-35(a) and the pedestrian movement scheme shown in Exhibit 19-35(b).

Exhibit 19-35
Crosswalk, Corner, and Movement Numbering Scheme for Two-Stage Crossings



(a) Crosswalk numbering scheme

(b) Traffic movement numbering scheme

With a two-stage crossing, the signal operation accommodates pedestrians crossing an intersection leg by providing pedestrian service during two signal phases. During the first phase, the pedestrians cross from the first corner to the median. During the second phase, they cross from the median to the next corner. The first phase to occur is denoted by the letter "X" and the second phase to occur is denoted by the letter "Y."

The crossing direction of interest and the phase sequence are considered to determine which phase is "Phase X" and which phase is "Phase Y". The two phase numbers of interest are identified in Exhibit 19-35(a) by the two-digit crosswalk number associated with the crosswalk of interest. For example, if the crosswalk between corner C and corner B is of interest, phases 1 and 2 are used to define Phase X and Phase Y. The crossing direction and phase sequence are considered in the following manner:

- If the crossing direction is clockwise (i.e., from corner B to corner C) and phase 1 leads phase 2 in the phase sequence, then Phase X is phase 1 and Phase Y is phase 2.
- If the crossing direction is clockwise (i.e., from corner B to corner C) and phase 1 lags phase 2 in the phase sequence, then Phase X is phase 2 and Phase Y is phase 2.
- If the crossing direction is counterclockwise (i.e., from corner C to corner B), then Phase X is phase 2 and Phase Y is phase 1 regardless of whether phase 1 leads or lags phase 2.

Required Data

The data needed for the procedure are identified in the following list:

- Cycle length (s);
- Phase sequence (list of phases in order of occurrence);
- Phase duration (sum of the duration of the green, yellow change, and red clearance intervals) for all phases (s);
- Walk interval duration for Phase X and Phase Y (s);
- Distance crossed during Phase X (i.e., distance from first corner to far side of median) (ft);
- Yellow change interval duration for Phase X and Phase Y (needed only if rest-in-walk is enabled or no pedestrian signal head provided) (s)
- Red clearance interval duration for Phase X and Phase Y (needed only if rest-in-walk is enabled or no pedestrian signal head provided) (s)
- Pedestrian clear duration for Phase X and Phase Y (needed only if phase is actuated and rest-in-walk is enabled) (s)

If the signal control is fully actuated, an average value is used for the cycle length and the green interval durations. If the signal control is semiactuated, an average value is used for the green interval duration of the actuated phases.

Step 1: Determine the Effective Walk Time

During this step, the analyst determines the effective walk time for Phase X and for Phase Y, as described in Step 1 of the core delay methodology. In the case of a two-stage crossing of a single intersection leg, for those crosswalk sections associated with two phases (i.e., the section has a two-digit number), time to cross the section is provided to pedestrians during one or both phases. If they are served during both phases, then an overlap is used. When using Equation 19-52 or Equation 19-53 for a crosswalk section served by two phases (i.e., when overlap is used), the duration of phase i $D_{p,i}$ used in either equation must equal the sum of the duration of both phases that are parent to the overlap. The yellow change, red clearance, and pedestrian clear values are equal to those for the parent phase that occurs last in the overlap pair. For example, when using Equation 19-52 to compute the effective walk time for crosswalk section 12, the variable $D_{p,i}$ in this equation must equal the sum of the durations for phases 1 and 2 (i.e., $D_{p,12} = D_{p,1} + D_{p,2}$).

Step 2. Determine Crossing Time during First Stage

The time required to cross from the first corner to the median is determined in this step. This time is computed using Equation 19-77.

Equation 19-77

$$t_X = \frac{L_X}{S_p}$$

where

t_X = time for pedestrians to cross during Phase X (s),

L_X = distance from the first corner to the far side of the median (measured along the path of the pedestrian crossing) (ft), and

S_p = average pedestrian crossing speed (ft/s).

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

In the absence of local data, an average pedestrian walking speed of 4.0 ft/s is recommended when less than 20 percent of the pedestrians are elderly (i.e., age 65 or older), and 3.3 ft/s otherwise (*26*).

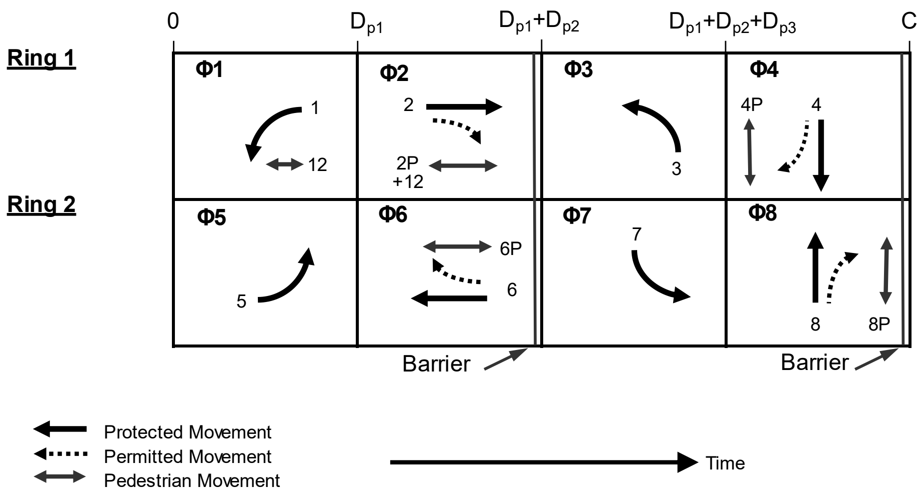
Step 3. Determine the Start of the Walk Intervals

During this step, the relative time in the cycle that the subject Walk intervals start is determined. Specifically, this is the start time for the Walk intervals associated with Phase X and Phase Y. To establish the relative start time for a given Walk interval T_{Walk}, one phase in the sequence will be established as time 0 (i.e., the start of the cycle). The start time of all subsequent phases will be established using the cumulative duration of the preceding phases.

With the relative phase start times established in this manner, the relative time for the start of a phase's Walk interval can be established by summing the preceding phase durations. In general, a Walk interval's relative start time is equal to its parent phase's relative start time. However, if a leading pedestrian interval is used, then the Walk interval's relative start time equals the phase relative start time minus the leading interval duration. Similarly, if a lagging pedestrian interval is used, then the Walk interval's relative start time equals the phase relative start time plus the lagging interval duration.

To illustrate the guidance provided for this step, consider an analysis of the pedestrian crossing from corner B to corner C in Exhibit 19-35(a), where the intersection has the phase sequence shown in Exhibit 19-33. Based on the numbering scheme shown in Exhibit 19-35(a), this crossing is served by phases 1 and 2. Based on Exhibit 19-36, phase 1 occurs first for the subject crossing direction (i.e., X = 1) and phase 2 occurs second (i.e., Y = 2). The start time of Phase X is 0. The start time of Phase Y is equal to the duration of phase 1 D_{p1}. Although not needed for this illustration, the start time for phase 3 is shown in Exhibit 19-36 to equal the sum of the phase 1 duration and phase 2 duration.

Exhibit 19-36
Example Phase Sequence for Two-Stage Crossing Shown Using Dual-Ring Structure



Both Walk intervals start with their parent phase for this illustration, so the relative start time of the Walk interval for Phase X $T_{\text{Walk},X}$ is 0 and that for Phase Y $T_{\text{Walk},Y}$ is equal to D_{p1}. Other values will likely be obtained for other phase sequences.

Continuing the illustration, consider an analysis of the pedestrian crossing from corner C to corner B in Exhibit 19-35(a), where the intersection has the phase sequence shown in Exhibit 19-33. Based on the numbering scheme shown in Exhibit 19-35(a), this crossing is served by phases 1 and 2. Based on Exhibit 19-36, phase 2 occurs first for the subject crossing direction (i.e., X = 2) and phase 1 occurs second (i.e., Y = 1). The start time of Phase X is equal to D_{p1}. The start time of Phase Y is equal to 0.

Step 4. Compute Delay for First-Stage Crossing

During this step, the delay for the first-stage crossing is that incurred by pedestrians waiting at the first corner. This delay is computed using Equation 19-78.

$$d_{p,1} = \frac{\left(C - g_{\text{Walk},x}\right)^2}{2\,C}$$

Equation 19-78

where

$d_{p,1}$ = pedestrian delay at corner for stage 1 (s/p),

C = cycle length (s), and

$g_{\text{Walk},X}$ = effective walk time for Phase X serving the subject pedestrian movement (s).

Step 5. Compute Delay for Second-Stage Crossing Given Arrival is during Don't Walk

During this step, the second-stage crossing delay is computed for one portion of the pedestrian stream. This particular delay is that incurred by pedestrians waiting *on the median* that arrived at the first corner during a Don't Walk indication (flashing or solid). The other portion of the second-stage crossing delay is computed in the next step.

A. Compute the Time between Walk Intervals. The time between the Walk intervals for Phases X and Y is computed using Equation 19-79.

Equation 19-79

$$t_{YX} = \text{Modulo}\left(T_{\text{Walk},Y} - T_{\text{Walk},X}, C\right)$$

where

t_{YX} = time between start of Walk intervals (s),

$T_{\text{Walk},X}$ = relative start time of the Walk interval for Phase X (s),

$T_{\text{Walk},Y}$ = relative start time of the Walk interval for Phase Y (s), and

C = cycle length (s).

The modulo function in Equation 19-79 ensures that the value for t_{YX} is a non-negative number that is less than the cycle length. When used, the equation in the parentheses is computed and the resulting value is compared to the range 0 to C. If this value is outside the range, the value is changed by adding (or subtracting) one cycle length and then range satisfaction reassessed. The value is changed by adding or subtracting additional cycle length increments until it is within the range 0 to C.

B. Compute the Delay Given Arrival is during Don't Walk. The delay is that incurred by pedestrians waiting *on the median* that arrived at the first corner during a Don't Walk indication is computed using the following equations.

Equation 19-80

$$d_{2,DW1} = \begin{cases} t & \text{if } t < C - g_{\text{Walk},Y} \\ 0 & \text{if } t \geq C - g_{\text{Walk},Y} \end{cases}$$

with

Equation 19-81

$$t = \text{Modulo}(t_{YX} - t_X, C)$$

where

$d_{2,DW1}$ = delay on median for stage 2, given arrival is during a Don't Walk indication at corner (s/p);

t = waiting time on median when pedestrians reach median during a Don't Walk indication (s);

$g_{\text{Walk},Y}$ = effective walk time for Phase Y serving the subject pedestrian movement (s);

t_X = time for pedestrians to cross during Phase X (s); and

all other variables are as previously defined.

Step 6. Compute Delay for Second-Stage Crossing Given Arrival is during Walk

During this step, the second-stage crossing delay is computed for the second portion of the pedestrian stream. This particular delay is that incurred by pedestrians waiting *on the median* that arrived at the first corner during the Walk indication.

There are two sets of equations that can be used to compute the second-stage crossing delay. The correct set of equations is determined by comparing the value of t with the effective walk time for Phase X. These two sets of equations are described in the following paragraphs.

When $t < g_{\text{Walk},X}$ compute the second-stage crossing delay using Equation 19-82 and Equation 19-83.

$$d_{2,W1} = \begin{cases} \dfrac{0.5(a+t)^2 + a\left(C - g_{\text{Walk},X}\right)}{g_{\text{Walk},X}} & \text{if } \left(t + g_{\text{Walk},Y}\right) < g_{\text{Walk},X} \\ \dfrac{0.5\,t^2}{g_{\text{Walk},X}} & \text{if } g_{\text{Walk},X} \le \left(t + g_{\text{Walk},Y}\right) \le C \\ \dfrac{0.5\left(C - g_{\text{Walk},Y}\right)^2}{g_{\text{Walk},X}} & \text{if } \left(t + g_{\text{Walk},Y}\right) > C \end{cases}$$

Equation 19-82

with

$$a = g_{\text{Walk},X} - g_{\text{Walk},Y} - t$$

Equation 19-83

where

$d_{2,W1}$ = delay on median for stage 2, given arrival is during the Walk indication at corner (s/p);

t = waiting time on median when pedestrians reach median during a Don't Walk indication (s);

a = undefined intermediate variable; and

all other variables are as previously defined.

When $t \ge g_{\text{Walk},X}$ compute the second-stage crossing delay using Equation 19-84 and Equation 19-85.

$$d_{2,W1} = \begin{cases} t - 0.5\,g_{\text{Walk},X} & \text{if } \left(t + g_{\text{Walk},Y}\right) < C \\ \dfrac{0.5\,b^2 + b\left(t - g_{\text{Walk},X}\right)}{g_{\text{Walk},X}} & \text{if } C \le \left(t + g_{\text{Walk},Y}\right) \le \left(C + g_{\text{Walk},X}\right) \\ 0 & \text{if } \left(t + g_{\text{Walk},Y}\right) > \left(C + g_{\text{Walk},X}\right) \end{cases}$$

Equation 19-84

with

$$b = g_{\text{Walk},X} - g_{\text{Walk},Y} - t + C$$

Equation 19-85

where

$d_{2,W1}$ = delay on median for stage 2, given arrival is during the Walk indication at corner (s/p);

b = undefined intermediate variable; and

all other variables are as previously defined.

Step 7. Compute Delay for Two-Stage Crossing

The pedestrian delay for a two-stage crossing is computed using the following equations.

Equation 19-86

$$d_p = d_{p,1} + \left[d_{2,DW1}P_{DW1} + d_{2,W1}(1 - P_{DW1})\right]$$

with

Equation 19-87

$$P_{DW1} = \frac{(C - g_{\mathrm{Walk,X}})}{C}$$

where

d_p = pedestrian delay (s/p);

$d_{p,1}$ = pedestrian delay at corner for stage 1 (s/p);

$d_{2,DW1}$ = delay on median for stage 2, given arrival is during a Don't Walk indication at corner (s/p);

$d_{2,W1}$ = delay on median for stage 2, given arrival is during the Walk indication at corner (s/p); and

P_{DW1} = proportion of arrivals during a Don't Walk indication at corner (decimal).

Crossing Two Intersection Legs in Two Stages

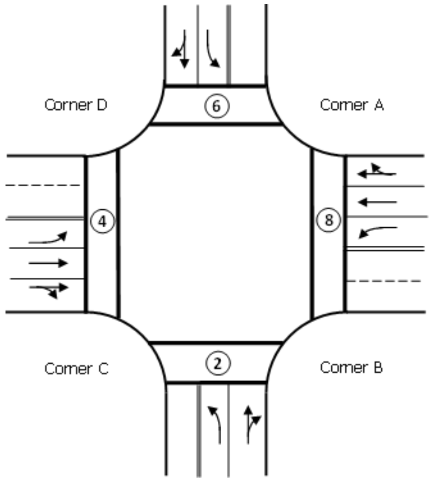
This procedure estimates the delay to pedestrians that cross two intersection legs during two phases of one cycle to complete a diagonal crossing of the intersection (*36*). The delay when crossing the first crosswalk is computed and the delay for crossing both crosswalks as a system is computed. The delay when crossing the second crosswalk is computed by subtracting the first crosswalk delay from the system delay.

A diagonal crossing at a typical four-leg intersection has two possible travel paths depending on whether the major street leg is crossed first or second. These two paths are referred to as the "clockwise path" and the "counterclockwise path." The procedure estimates the delay to a given path of travel when crossing from one corner to the diagonally opposite corner using two crosswalks. The procedure is applied separately to evaluate the other travel path between the two diagonal corners or to evaluate diagonal crossings for other corner combinations. Guidance on evaluating a direct diagonal crossing as part of an exclusive pedestrian phase (e.g., Barnes dance, pedestrian scramble) is provided below.

The procedure is based on the following two assumptions: (*a*) pedestrian arrivals to the first crossing location (i.e., street corner) are random, and (*b*) they will begin their crossing during the first available Walk interval (regardless of whether it is to cross the minor street leg or the major street leg). Assumption "b" reflects the pedestrian's desire to minimize their total diagonal crossing delay.

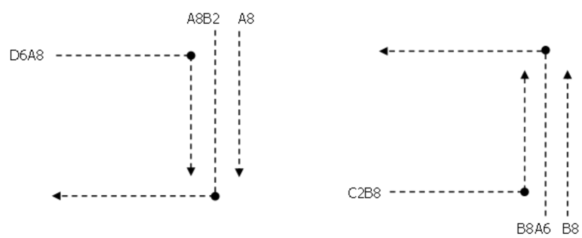
The procedure described in this section is based on the vehicle movement numbering scheme shown in Exhibit 19-31(b). These vehicle movement numbers correspond to the signal phase that serves the movement (i.e., vehicle movement 2 is served by signal phase 2), which follows the traditional eight-phase dual-ring structure. The procedure is also based on the crosswalk numbering scheme shown in Exhibit 19-37(a) and the pedestrian movement schemes shown in Exhibit 19-37(b–e).

Exhibit 19-37
Crosswalk, Corner, and Movement Numbering Scheme for Two-Stage Diagonal Crossings

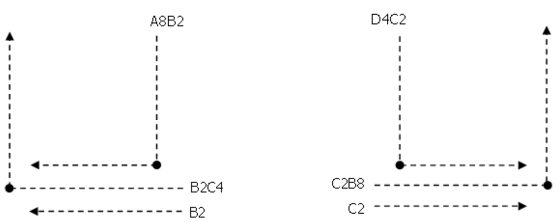


(a) Crosswalk numbering scheme

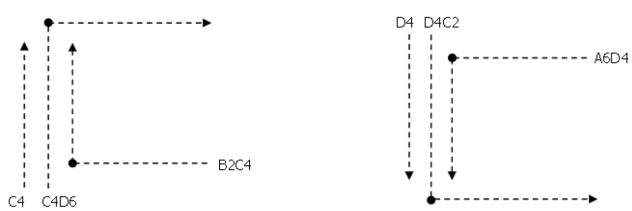
(b) Pedestrian movement numbering for crosswalk 8



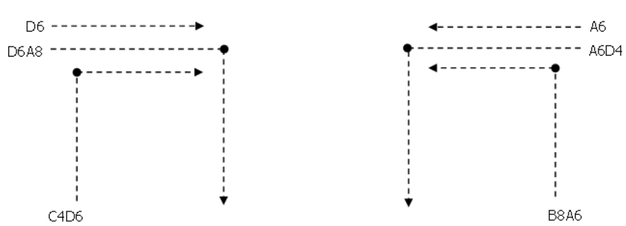
(c) Pedestrian movement numbering for crosswalk 2



(d) Pedestrian movement numbering for crosswalk 4



(e) Pedestrian movement numbering for crosswalk 6



With a diagonal crossing, the signal operation accommodates pedestrians crossing to the diagonally opposite corner by providing pedestrian service during two signal phases. During the first phase, the pedestrians cross from the first corner to the second corner. During the second phase, they cross from the second corner to the last corner.

The delay incurred during a diagonal crossing is dependent on the direction the pedestrian travels around the intersection (i.e., clockwise or counterclockwise). As a result, the direction of interest must be specified when using the procedure. The first phase to occur in the subject travel direction is denoted by the letter "X." The second phase to occur in the subject direction of travel is denoted by the letter "Y." Had the pedestrian decided to cross in the other direction around the intersection, two different signal phases would provide pedestrian service. The first phase to serve travel in the other direction is denoted by the letter "Z" (i.e., Phase Z is the first phase to serve the pedestrian starting the diagonal crossing in a direction *opposite* to the direction of interest).

To illustrate the aforementioned rules, consider the intersection shown in Exhibit 19-37(a). An analyst desires to compute the delay to a pedestrian traveling in a clockwise path from corner B to D. Based on this information, the first phase to serve the pedestrian crossing in the subject travel direction (i.e., from corner B to corner C) is phase 2, so Phase X is phase 2 (i.e., X = 2). The second phase to serve pedestrians in the subject travel direction (i.e., from corner C to corner D) is phase 4, so Phase Y is phase 4 (i.e., Y = 4). If the pedestrian were to travel in the other direction, phase 8 would be the first phase to provide service (i.e., from corner B to corner A), so Phase Z is phase 8 (i.e., Z = 8).

Required Data

The data needed for the procedure are as follows:

- Cycle length (s)
- Phase sequence (list of phases in order of occurrence)
- Phase duration (sum of the duration of the green, yellow change, and red clearance intervals) for all phases (s)
- Walk interval duration for Phase X and Phase Z (s)
- Distance crossed during Phase X (i.e., distance from first corner to second corner) (ft)
- Yellow change interval duration for Phase X and Phase Z (needed only if rest-in-walk is enabled or no pedestrian signal head provided) (s)
- Red clearance interval duration for Phase X and Phase Z (needed only if rest-in-walk is enabled or no pedestrian signal head provided) (s)
- Pedestrian clear duration for Phase X and Phase Z (only needed if phase is actuated and rest-in-walk is enabled) (s)

If the signal control is fully actuated, an average value is used for the cycle length and the green interval durations. If the signal control is semiactuated, an average value is used for the green interval duration of the actuated phases.

Step 1. Determine the Effective Walk Time

During this step, the analyst determines the effective walk time for Phase X and for Phase Y, as described in Step 1 of the core pedestrian delay methodology.

Step 2. Determine Crossing Time during First Phase

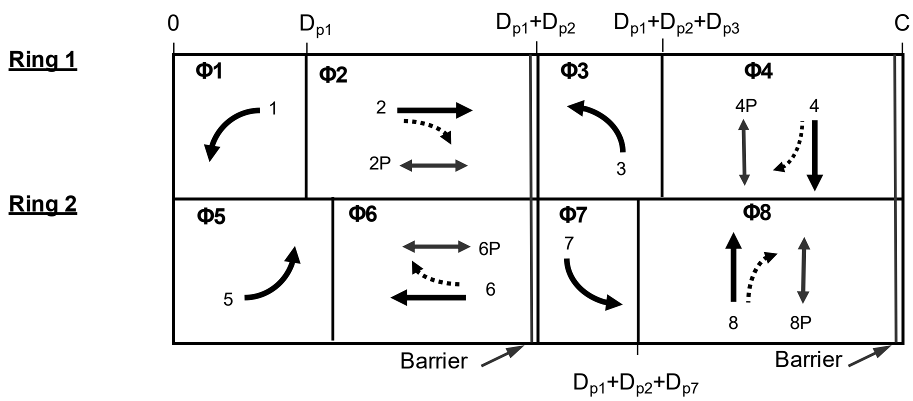
The time required to cross from the first corner to the second corner is determined in this step. This step is identical to Step 2 for the procedure for crossing two intersection legs in one stage, except that the crossing length L_X is defined as the distance from the first corner to the second corner, in feet, measured along the path of the pedestrian crossing.

Step 3. Determine the Start of the Walk Intervals

During this step, the relative time in the cycle that the subject Walk intervals start is determined. Specifically, this is the start time for the Walk intervals associated with Phase X, Phase Y, and Phase Z. This step is performed similarly to Step 3 for crossing one intersection leg in two stages.

To illustrate the guidance provided for this step, consider an analysis of the clockwise diagonal crossing from corner B to corner D in Exhibit 19-37(a), where the intersection has the phase sequence shown in Exhibit 19-38. Based on the numbering scheme shown in Exhibit 19-37(a), the subject travel direction is served first by phase 2 and then phase 4. Had a counterclockwise crossing been taken, the diagonal crossing would be served first by phase 8.

Exhibit 19-38
Example Phase Sequence for Two-Stage Diagonal Crossing Shown Using Dual-Ring Structure



Based on Exhibit 19-38, phase 2 occurs first for the subject travel direction (i.e., X = 2) and phase 4 occurs second for the subject direction of travel (i.e., Y = 4). Phase 8 occurs first for the other travel direction (i.e., Z = 8). The start time of Phase X is equal to the duration of phase 1 D_{p1} (since phase 2 starts when phase 1 ends for the sequence shown in Exhibit 19-38). The start time of Phase Y is equal to the duration of phases 1, 2, and 3 (= $D_{p1} + D_{p2} + D_{p3}$) (since phase 4 starts when phase 3 ends for the sequence shown Exhibit 19-38). Similarly, the start time of Phase Z is equal to the duration of phases 1, 2, and 7 (= $D_{p1} + D_{p2} + D_{p7}$). Note that the dual-ring structure shown in Exhibit 19-38 has a barrier at the end of phases 2 and 6 which requires the duration of phase 1 plus phase 2 to equal the duration of phase 5 plus phase 6.

Both Walk intervals start with their parent phase for this illustration, so the relative start time of the Walk interval for Phase X $T_{\text{Walk},X}$ is D_{p1}, Phase Y $T_{\text{Walk},Y}$ is $D_{p1} + D_{p2} + D_{p3}$ and that for Phase Z $T_{\text{Walk},Z}$ is $D_{p1} + D_{p2} + D_{p7}$. Other values will likely be obtained for other phase sequences.

Step 4. Compute Delay for First-Stage Crossing

During this step, the delay for the first-stage crossing in the subject travel direction is computed. This delay is incurred by pedestrians that have been waiting at the first corner since the end of the effective walk time for the other travel direction.

A. Compute the End of Effective Walk Time. The end of the effective walk time for Phase X is computed using Equation 19-88.

Equation 19-88

$$T_X = \text{Modulo}\left(T_{\text{Walk},X} + g_{\text{Walk},X}, C\right)$$

where

T_X = relative end time of the effective walk period for Phase X (s),

$T_{\text{Walk},X}$ = relative start time of the Walk interval for Phase X (s),

C = cycle length (s), and

$g_{\text{Walk},X}$ = effective walk time for Phase X serving the subject pedestrian movement (s).

The modulo function was described with Step 5 of the procedure for crossing one intersection leg in two stages. The end of the effective walk time for Phase Z is computed using Equation 19-89.

Equation 19-89

$$T_Z = \text{Modulo}\left(T_{\text{Walk},Z} + g_{\text{Walk},Z}, C\right)$$

where

T_Z = relative end time of the effective walk period for Phase Z (s),

$T_{\text{Walk},Z}$ = relative start time of the Walk interval for Phase Z (s),

C = cycle length (s), and

$g_{\text{Walk},Z}$ = effective walk time for Phase Z serving the subject pedestrian movement (s).

B. Compute the Delay for the First-Stage Crossing. The delay for the first-stage crossing is computed using the following equations.

Equation 19-90

$$d_{p,1} = \frac{\left(t_{XZ} - g_{\text{Walk},X}\right)^2}{2\, t_{XZ}}$$

with

Equation 19-91

$$t_{XZ} = \text{Modulo}(T_X - T_Z, C)$$

where

$d_{p,1}$ = pedestrian delay at corner for stage 1 (s/p),

t_{XZ} = time between end of effective walk time for Phase Z and start of effective walk time for Phase X (s), and

all other variables are as previously defined.

Step 5. Compute Delay for Entire Diagonal Crossing

The delay for the entire diagonal crossing in the subject travel direction is computed in this step. This delay represents the sum of the delay incurred on the first corner and that incurred on the second corner. The delay for just the second stage crossing is computed in Step 6.

The diagonal crossing delay is computed using the following equations.

Equation 19-92

$$d_p = t_d - t_X$$

with

Equation 19-93

$$t_d = \begin{cases} T_{\text{Walk},Y} - \dfrac{T_X + T_Z}{2} & \text{if } T_{\text{Walk},Y} \geq T_X \geq T_Z \\ T_{\text{Walk},Y} - \dfrac{T_X + T_Z - C}{2} & \text{if } T_X < T_Z \\ T_{\text{Walk},Y} - \dfrac{T_X + T_Z}{2} + C & \text{if } T_X \geq T_Z \geq T_{\text{Walk},Y} \end{cases}$$

where

d_p = pedestrian delay (s/p),

t_d = time between arrival to first corner and departure from second corner (s),

t_X = time for pedestrians to cross during Phase X (s), and

$T_{\text{Walk},Y}$ = relative start time of the Walk interval for Phase Y (s), and

all other variables are as previously defined.

Step 6. Compute Delay for Second-Stage Crossing

During this step, the delay for the second-stage crossing in the subject travel direction is computed. This delay is incurred by pedestrians waiting at the second corner. It is computed using Equation 19-94.

Equation 19-94

$$d_{p,2} = d_p - d_{p,1}$$

where

$d_{p,2}$ = pedestrian delay at corner for stage 2 (s/p),

d_p = pedestrian delay (s/p), and

$d_{p,1}$ = pedestrian delay at corner for stage 1 (s/p).

Crosswalk Delay

The procedure for crossing two intersection legs in two stages describes the evaluation of a given diagonal pedestrian movement. However, this procedure can also be used to evaluate all movements associated with a given crosswalk (*37*). As shown in Exhibit 19-37(b–e), there are six pedestrian movements associated with each crosswalk. That is, each crosswalk has two directions of travel and each direction of travel is associated with three pedestrian movements.

Referring to Exhibit 19-37(a), consider crosswalk 2 and pedestrians crossing from corner C to B. As shown in Exhibit 19-37(c), the following pedestrian movements are of interest: D4C2, C2B8, and C2. Movement D4C2 represents the pedestrians that are completing a counterclockwise diagonal crossing from corner D to corner B. Their delay in crosswalk 2 can be estimated as the second-stage crossing delay of the diagonal crossing procedure (i.e., Equation 19-94).

Movement C2B8 represents the pedestrians that are completing a counterclockwise diagonal crossing from corner C to corner A. Their delay can be estimated as the first-stage crossing delay of the diagonal crossing procedure (i.e., Equation 19-90).

Finally, movement C2 represents the pedestrians that are crossing from corner C to corner B and are not destined for any other intersection corner. Their delay can be estimated using the core pedestrian delay procedure (Equation 19-54).

If the volume of each of these three movements is known, they can be used to compute a volume-weighted average delay for the subject travel direction of the crosswalk.

The process outlined in the preceding paragraphs can be repeated to evaluate the three pedestrian movements for the opposing travel direction of the subject crosswalk. A volume-weighted average delay for this travel direction can also be computed if the pedestrian volume is known for each of the three pedestrian movements.

Finally, if both travel directions of a given crosswalk have been evaluated to produce a delay for each of the six pedestrian movements and the volume of these six movements are known, then a volume-weighted average delay can be computed for the crosswalk.

Crosswalk Closure

This method is a logical extension of the two-leg/two-stage procedure, but has not been validated through field measurements or simulation.

The procedure for crossing two intersection legs in two stages can also be used to evaluate the delay associated with a crosswalk closure, in which pedestrians must cross three intersection legs to make their desired crossing (*37*). Referring to Exhibit 19-37(a), assume crosswalk 6, connecting corner D to A, is closed. To get from corner D to A, pedestrians must cross from D to C, C to B, and finally B to A.

To estimate the delay for the three-stage crossing, the procedure for crossing two intersection legs in two stages can be applied twice: once for the crossing from corner D to B, and again for the crossing from corner C to A. The delay for the three-stage crossing from corner D to A can be estimated as the delay to cross in two stages from corner D to B (Equation 19-92) plus the second-stage crossing delay to cross in two stages from corner C to A (Equation 19-94).

The delay associated with a three-stage crossing can be compared to the delay with making a direct, one-stage crossing (i.e., if the crosswalk was open) to estimate the added pedestrian delay associated with the crosswalk closure.

Exclusive Pedestrian Phase

An exclusive pedestrian phase, also known as a Barnes dance or pedestrian scramble, provides a signal phase in which only pedestrian movements are served and pedestrians can cross directly from one corner to any other corner, including diagonally opposite ones. In some implementations, pedestrians are only served during the exclusive pedestrian phase. In other implementations, crosswalks are also served during the parallel vehicular phases.

This method is a logical extension of the one-leg/one-stage and two-leg/two-stage procedures, but has not been validated through field measurements or simulation.

When pedestrians may only cross the intersection during the exclusive pedestrian phase, their delay can be calculated using the core pedestrian delay method (Equation 19-54). When crosswalks are also served during the parallel vehicular phases, the delay to cross to the opposite intersection corner can be estimated as the lower of (*a*) the delay to wait to make a direct diagonal crossing, using the core pedestrian delay method, and (*b*) the delay associated with making a two-stage crossing, using the procedure for crossing two intersection legs in two stages (*37*).

Coordinated Actuated Signal Operation with Permissive Period

This section describes a procedure for computing the delay to pedestrians crossing the major street at a signalized intersection that is part of a coordinated actuated signal system (*37*). The procedure is based on consideration of the potential delay that would be incurred if the pedestrian's arrival occurred at different times in the signal cycle. This delay is influenced by the minor street traffic movements that are monitored by detectors. It is also influenced by calls from the detectors serving pedestrians desiring to cross the major street. Hereafter, any reference to "calls" for minor street service is implied to mean the combined calls from these vehicle and pedestrian detectors.

This method has not been validated through field measurements or simulation.

In this situation, if a minor street call is received during the permissive period PP while the major street phase is active, the major street phase is terminated and the call is served following the major street pedestrian clear interval (or the yellow change and red clearance intervals if there is no pedestrian phase associated with the major street or rest in walk is not enabled for the major street). In this situation, the pedestrian delay is equal to the length of the major street pedestrian clear interval (or the yellow change and red clearance interval if the conditions stated above are met). If the minor street call occurs outside the permissive period while the major street phase is active, the minor street phase is not served until the next cycle.

An added complication is that a pedestrian call for the crosswalk or a vehicle call for the corresponding minor-street phase (if present) may have already occurred during the prior signal cycle, in which case the major street phase would have already terminated by the time the pedestrian arrived. If this were to happen every cycle (i.e., due to high side-street volumes, high crosswalk volumes, or both), the signal would act like a pre-timed signal for the crosswalk, and the pedestrian delay would be the same as in the core pedestrian delay method (Equation 19-54).

The pedestrian delay is therefore a function of (*a*) the probability P_{PC} that a prior call has been placed that would have activated the minor street phase at the start of the cycle, and (*b*) the length of the permissive period, if no prior call was placed. Calculating the overall delay requires looking at two consecutive cycles and calculating (*a*) the probability of a call being placed in the prior cycle that would have activated the minor street phase at the start of the current cycle and, if no call was placed, (*b*) the respective probabilities of a call being received during the pedestrian clear interval, the permissive period, and after the permissive period.

Pedestrians and minor street vehicles are assumed to arrive randomly and the calls associated with their arrival can be represented by a Poisson distribution. The probability that at least one minor street call is received during two consecutive cycles P_{2PC} is given by Equation 19-95.

Equation 19-95

$$P_{2PC} = 1 - e^{-(2C)q}$$

where C is the cycle length in seconds and q is the arrival rate of minor-street vehicles and pedestrians (in vehicles+persons per second) that can trigger a call that would terminate the major street phase, equal to the hourly vehicle and pedestrian volume divided by 3,600 s/h.

The probability P_{PC} of receiving one or more prior calls during the cycle duration C is given by Equation 19-96:

Equation 19-96

$$P_{PC} = 1 - e^{-Cq}$$

As two consecutive cycles are being evaluated, the individual probabilities of a call happening during a specific period within the two cycles need to be normalized by the probability of receiving at least one call during two consecutive cycles. This is done to accurately reflect event probabilities during low-volume conditions where successive calls may occur more than two cycles apart. The probability $P_{PC|2}$ of receiving one or more calls during the cycle duration C, given that one or more calls are received during two consecutive cycles, is given by Equation 19-97:

Equation 19-97

$$P_{PC|2} = \frac{P_{PC}}{P_{2PC}}$$

The probability $P_{PCI|2}$ of receiving at least one call during the pedestrian clearance interval, given that one or more calls are received during two consecutive cycles is the normalized probability that at least one call is received during the prior cycle duration plus the pedestrian clearance interval for the major street phase PC_{mj}, minus the normalized probability that at least one call is received during the prior cycle, as shown by Equation 19-98:

Equation 19-98

$$P_{PCI|2} = \frac{1 - e^{-(C + PC_{mj})q}}{P_{2PC}} - P_{PC|2}$$

The probability $P_{PP\text{-}PCI|2}$ of receiving at least one call during the permissive period, given that one or more calls are received during two consecutive cycles is the normalized probability that at least one call is received during the prior cycle plus the greater of the pedestrian clearance interval PC_{mj} or the permissive period PP, minus the normalized probability that at least one call is received during the prior cycle or pedestrian clearance interval, as shown in Equation 19-99:

$$P_{PP-PCI|2} = \frac{1 - e^{-(C + \max[PP, PC_{mj}])q}}{P_{2PC}} - \frac{1 - e^{-(C + PC_{mj})q}}{P_{2PC}}$$

Equation 19-99

Finally, the probability $P_{C\text{-}PP|2}$ of receiving at least one call following the current cycle's permissive period, given that one or more calls are received during two consecutive cycles is 1 minus the normalized probability that at least one call is received during the prior cycle or by the end of the permissive period or pedestrian clearance interval (whichever is greater) in the current cycle, as shown in Equation 19-100:

$$P_{C-PP|2} = 1 - \frac{1 - e^{-(C + \max[PP, PC_{mj}])q}}{P_{2PC}}$$

Equation 19-100

The average pedestrian delay $d_{ped,i}$ for pedestrian movement *i* is then the probability-weighted average of (*a*) the average delay when a prior call has been placed, (*b*) the delay with no prior call and a call occurs during the pedestrian clear interval, (*c*) the delay with no prior call and a call occurs during the time period after the pedestrian clear interval but before the permissive period ends, and (*d*) the average delay with no prior call and a call occurs outside the permissive period, as given by Equation 19-68:

$$d_{ped,i} = P_{PC|2}\, d_{PC,i} + P_{PCI|2}\, d_{PCI,i} + P_{PP-PCI|2}\, d_{PP-PCI,i} + P_{C-PP|2}\, d_{C-PP,i}$$

Equation 19-101

where the average delays associated with each situation are given by Equation 19-102 through Equation 19-105:

$$d_{PC,i} = \frac{\left(C - g_{\text{Walk},i}\right)^2}{2\,C}$$

Equation 19-102

$$d_{PCI,i} = \left(\frac{PC_{mj}}{C}\right)\left(\frac{PC_{mj}}{2} + YAR_{mj}\right) + \frac{\left(C - PC_{mj} - g_{\text{Walk},i}\right)^2}{2\,C}$$

Equation 19-103

$$d_{PP-PCI,i} = \left(\frac{\max[PP, PC_{mj}] - PC_{mj}}{C}\right) YAR_{mj} + \frac{\left(C - \max[PP, PC_{mj}] - g_{\text{Walk},i}\right)^2}{2\,C}$$

Equation 19-104

$$d_{C-PP,i} = \frac{\left(C - \max[PP, PC_{mj}] - g_{\text{Walk},i}\right)^2}{2\,C}$$

Equation 19-105

where

$d_{PC,i}$ = average delay for pedestrian movement *i* when a prior call has been placed (s),

$d_{PCI,i}$ = average delay for pedestrian movement *i* with no prior call and a call occurs during the pedestrian clear interval (s),

PC_{mj} = pedestrian clearance interval for the major street phase (s),

CP_{mj} = sum of yellow change and red clearance intervals for the major street phase (s),

$d_{PP-PCI,i}$ = average delay for pedestrian movement i with no prior call and a call occurs during the time period after the pedestrian clear interval but before the permissive period ends (s),

PP = permissive period length (s),

$d_{C-PP,i}$ = average delay for pedestrian movement i with no prior call and a call occurs outside the permissive period (s), and

all other variables as defined previously.

This method assumes that the sum of the permissive period, the major street minimum green, and the major and minor street pedestrian clearance intervals (or yellow change and red clearance interval, as appropriate) is less than or equal to the cycle length.

Free Operation

This method has not been validated through field measurements or simulation.

In this scenario, the cycle length is variable, as phases terminate once their demand has been served or the maximum phase length has been reached. The signalized intersection method for motorized vehicles (Section 3.) accommodates this scenario (*37*) and calculates an average cycle length C on the basis of the vehicular and pedestrian demand on each approach. This cycle length is then used with Equation 19-54 in the core pedestrian delay method to calculate average pedestrian delay for a pedestrian movement.

6. BICYCLE METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to bicyclists traveling through a signalized intersection.

SCOPE OF THE METHODOLOGY

The overall scope of the three methodologies was provided in Section 2. This section identifies the additional conditions applicable to the bicycle methodology.

- *Target travel modes*. The bicycle methodology addresses travel by bicycle through a signalized intersection. It is not designed to evaluate the performance of other travel means (e.g., motorized bicycle, rickshaw).
- *Shared or exclusive lanes*. The bicycle methodology can be used to evaluate the service provided to bicyclists when sharing a lane with motorized vehicles or when traveling in an exclusive bicycle lane.

Spatial Limits

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

Performance Measures

Performance measures applicable to the bicycle travel mode include bicycle delay and bicycle LOS score. The LOS score is an indication of the typical bicyclist's perception of the overall crossing experience.

LOS is also considered a performance measure. It is useful for describing intersection performance to elected officials, policy makers, administrators, or the public. LOS is based on the bicyclist LOS score.

Limitations of the Methodology

This subsection identifies the known limitations of the bicycle methodology. If one or more of these limitations is believed to have an important influence on the performance of a specific intersection, then the analyst should consider using alternative methods or tools for the evaluation.

The bicycle methodology does not account for the effect of the following conditions on the quality of service provided to bicyclists:

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

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the bicycle methodology. These data are listed in Exhibit 19-39. The second column (labeled Basis) of the exhibit indicates whether the input data are needed for each signal phase, each intersection approach, or the intersection as a whole. The exhibit also lists default values that can be used if local data are not available (*25*, *30*).

Exhibit 19-39
Required Input Data, Potential Data Sources, and Default Values for Bicycle Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
Traffic Characteristics			
Motorized vehicle demand flow rate (veh/h)	A	Field data, past counts	Must be provided
Bicycle flow rate (bicycles/h)	A	Field data, past counts	Must be provided
Proportion of on-street parking occupied (decimal)	A	Field data	0.50 (if parking lane present)
Geometric Design			
Street width (ft)	A	Field data, aerial photo	Based on a 12-ft lane width
Number of lanes	A	Field data, aerial photo	Must be provided
Width of outside through lane (ft)	A	Field data, aerial photo	12 ft
Width of bicycle lane (ft)	A	Field data, aerial photo	5.0 ft (if provided)
Width of paved outside shoulder (ft)	A	Field data, aerial photo	Must be provided
Width of striped parking lane (ft)	A	Field data, aerial photo	8.0 ft (if present)
Signal Control Data			
Cycle length (s)	I	Field data	Same as motorized vehicle mode
Yellow change + red clearance (s)[a]	P	Field data	4 s
Duration of phase serving bicycles (s)	P	Field data	Same as motorized vehicle mode
Other Data			
Analysis period duration (h)[b]	I	Set by analyst	0.25 h

Notes: A = approach: one value for the intersection approach.
I = intersection: one value for the intersection.
P = phase: one value or condition for each signal phase.
[a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.
[b] Analysis period duration is as defined for Exhibit 19-11.

The data elements listed in Exhibit 19-39 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during presentation of the methodology.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 19-39. These data describe the traffic streams traveling through the intersection during the study period. The demand flow rate of motorized vehicles and bicycle flow rate are defined in Section 3 for the motorized vehicle mode.

The variable for the proportion of on-street parking occupied represents the proportion of the intersection's right-side curb line that has parked vehicles present during the analysis period. It is based on a zone that extends from a point 250 ft upstream of the intersection to the intersection and a second zone that extends from the intersection to a point 250 ft downstream of the intersection. If parking is not allowed in these two zones, then this proportion equals 0.0.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 19-39. These data describe the geometric elements that influence intersection performance from a bicyclist perspective. The number-of-lanes variable is defined in Section 3 for the motorized vehicle mode.

Street Width

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

Widths of Outside Through Lane, Bicycle Lane, Outside Shoulder, and Parking Lane

The widths of several individual elements of the cross section are considered input data. These elements include the outside lane that serves motorized vehicles at the intersection, the bicycle lane adjacent to the outside lane (if used), paved outside shoulder, and striped parking lane.

The outside lane width does not include the width of the gutter. If curb and gutter are present, then the width of the gutter is included in the shoulder width (i.e., shoulder width is measured to the curb face when a curb is present).

Signal Control Data

This subsection describes the data in Exhibit 19-39 that are identified as signal control. The yellow change interval and red clearance interval settings are defined in Section 3 for the motorized vehicle mode.

Cycle Length

Cycle length is predetermined for pretimed or coordinated-actuated control. Chapter 31 provides a procedure for estimating a reasonable cycle length for these two types of control when cycle length is unknown. Default values for cycle length are defined in Section 3 of the present chapter for the motorized vehicle mode.

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

Duration of Phase Serving Pedestrians and Bicycles

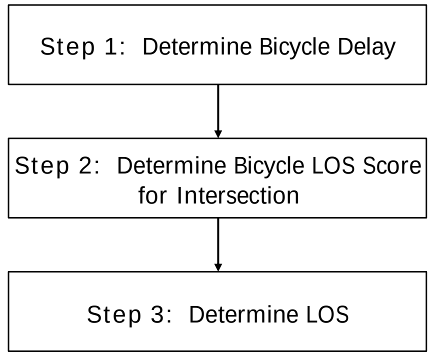
The duration of each phase that serves a bicycle movement is a required input. This phase is typically the phase that serves the through movement that is adjacent to the sidewalk and for which the bicycle and through vehicle travel paths are parallel. For example, Phases 2, 4, 6, and 8 are the phases serving the bicycle movements in Exhibit 19-2.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to bicyclists. The methodology is applied through a series of three steps that determine the bicycle LOS for an intersection approach. These steps are illustrated in Exhibit 19-40.

The methodology is focused on analyzing signalized intersection performance from the bicyclist point of view. Chapter 18, Urban Street Segments, describes a methodology for evaluating urban street performance with respect to the bicycle mode.

Exhibit 19-40
Bicycle Methodology for Signalized Intersections



COMPUTATIONAL STEPS

Step 1: Determine Bicycle Delay

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

Bicycle delay can be calculated only for intersection approaches that have an on-street bicycle lane or a shoulder that can be used by bicyclists as a bicycle lane. Bicyclists who share a lane with motorized vehicle traffic will incur the same delay as the motorized vehicles.

A. Compute Bicycle Lane Capacity

A wide range of capacities and saturation flow rates have been reported by many countries for bicycle lanes at intersections. Research indicates the base saturation flow rate may be as high as 2,600 bicycles/h (*38*). However, few intersections provide base conditions for bicyclists, and current information is insufficient to calibrate a series of appropriate saturation flow adjustment factors. Until such factors are developed, it is recommended that a saturation flow rate of 2,000 bicycles/h be used as an average value achievable at most intersections.

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

The capacity of the bicycle lane at a signalized intersection may be computed with Equation 19-106.

Equation 19-106

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

where

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

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

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

C = cycle length (s).

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

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 19-107.

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

Equation 19-107

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

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

At most signalized intersections, the only delay to through bicycles is caused by the signal, because bicycles have the right-of-way over right-turning vehicles during the green indication. Bicycle delay could be longer than that obtained from Equation 19-107 when (*a*) bicycles are forced to weave with right-turning traffic during the green indication or (*b*) drivers do not acknowledge the bicycle right-of-way because of high flows of right-turning vehicles.

The delay obtained from Equation 19-107 can be used to make some judgment about intersection performance. Bicyclists tend to have about the same tolerance for delay as pedestrians. They tend to become impatient when they experience a delay in excess of 30 s/bicycle. In contrast, they are very likely to comply with the signal indication if their expected delay is less than 10 s/bicycle.

Step 2: Determine Bicycle LOS Score for Intersection

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

The bicycle LOS score for the intersection $I_{b,int}$ is calculated by using Equation 19-108 through Equation 19-111.

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

Equation 19-108

with

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

Equation 19-109

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

Equation 19-110

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

Equation 19-111

where

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

F_w = cross-section adjustment factor;

F_v = motorized vehicle volume adjustment factor;

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

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

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

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

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

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

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

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

W_{pk} = width of striped parking lane (ft);

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

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

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

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

Step 3: Determine LOS

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

The bicycle LOS is determined by using the bicycle LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 19-9 to determine the LOS for the subject approach.

7. APPLICATIONS

EXAMPLE PROBLEMS

Chapter 31, Signalized Intersections: Supplemental, describes the application of each of the three methodologies through the use of example problems. There is one example problem associated with each core modal methodology, plus two example problems illustrating extensions to the core pedestrian delay method. The examples illustrate the operational analysis type.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 19-41 shows an illustrative generalized service volume table for a signalized intersection. This particular exhibit has been prepared for illustrative purposes only and should not be used for any specific planning or preliminary engineering application because the values in the table are highly dependent on the assumed input variables. Care must be taken in constructing a table that the analyst believes is representative of a typical signalized intersection within the planning area. In the example table, the volumes represent the total approach volume (sum of the left-, through, and right-turn movements). This particular table illustrates how hourly service volumes vary with the number of through lanes on the approach and the through movement *g*/*C* ratio.

Exhibit 19-41
Illustrative Generalized Service Volumes for Signalized Intersections (veh/h)

Through Movement *g/C* Ratio	No. of Through Lanes	LOS B	LOS C	LOS D	LOS E
0.40	1	130	610	730	800
	2	270	1,220	1,430	1,550
	3	380	1,620	1,980	2,000
0.45	1	320	720	840	910
	2	630	1,410	1,610	1,740
	3	840	1,780	2,000	2,250
0.5	1	490	830	940	1,020
	2	940	1,580	1,790	1,930
	3	1,180	1,930	2,000	2,500

Notes: LOS E threshold is defined by control delay greater than 80 s/veh or volume-to-capacity ratio >1.0.
Assumed values for all entries:
Heavy vehicles: 0%
Peak hour factor: 0.92
Lane width: 12 ft
Grade: 0%
Separate left-turn lane: yes
Separate right-turn lane: no
Pretimed control
Cycle length: 90 s
Lost time: 4 s/phase
Protected left-turn phasing: yes
g/C ratio for left-turn movement: 0.10
Parking maneuvers per hour: 0
Buses stopping per hour: 0
Percentage left turns: 10%
Percentage right turns: 10%

The hourly service volumes could easily be converted to daily service volumes with the application of appropriate *K*- and *D*-factors. Step-by-step instructions are provided in Appendix B of Chapter 6, HCM and Alternative

Analysis Tools, for users wishing to learn more about constructing one's own service volume table.

ANALYSIS TYPE

The three methodologies described in this chapter can each be used in three types of analysis. These analysis types are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis type are described in the subsequent paragraphs.

Operational Analysis

The objective of an operational analysis is to determine the LOS for current or near-term conditions when details of traffic volumes, geometry, and traffic control conditions are known. All the methodology steps are implemented and all calculation procedures are applied for the purpose of computing a wide range of performance measures. The operational analysis type will provide the most reliable results because it uses no (or minimal) default values.

Design Analysis

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

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

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

Planning and Preliminary Engineering Analysis

The objective of a planning and preliminary engineering analysis can be (*a*) to determine the LOS for either a proposed intersection or an existing intersection in a future year or (*b*) to size the overall geometrics of a proposed intersection.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses because default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose are described above in the sections associated with each methodology.

The requirement for a complete description of the signal-timing plan can be a burden for some planning analyses. The planning-level analysis application described in Chapter 31, Signalized Intersections: Supplemental, can be used to estimate a reasonable timing plan, in conjunction with the default values provided.

USE OF ALTERNATIVE TOOLS

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

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This subsection contains specific guidance for the application of alternative tools to the analysis of signalized intersections. Additional information on this topic is provided in the Technical Reference Library in Volume 4. The focus of this subsection is the application of alternative tools to evaluate motorized vehicle operation.

Comparison of Motorized Vehicle Methodology and Alternative Tools

Motorized Vehicle Methodology

The motorized vehicle methodology models the driver–vehicle–road–signal system with reasonable accuracy for most applications. It accounts for the operation of actuated phases, shared lanes, and permitted turn movements. It can account for the effect of initial queues and signal progression on delay.

The motorized vehicle methodology offers several advantages over alternative analysis tools. One advantage is that it has an empirically calibrated procedure for estimating saturation flow rate. Alternative tools often require saturation flow rate as an input variable. A second advantage is that it produces a direct estimate of capacity and volume-to-capacity ratio. These measures are not directly available from simulation tools. A third advantage is that it produces an expected value for each of several performance measures in a single application. Simulation tools require multiple runs and manual calculations to obtain an expected value for a given performance measure. A fourth advantage is that its analytic procedures are described in the HCM so that analysts can understand the driver–vehicle–road interactions and the means by which they are modeled. Most proprietary alternative tools operate as a “black box,” providing little detail describing the intermediate calculations.

Alternative Tools

Deterministic tools and simulation tools are in common use as alternatives to the motorized vehicle methodology offered in this chapter. Deterministic tools are often used for the analysis of signalized intersections. The main reasons for their popularity are found in the user interface, optimization options, and output presentation features. Some also offer additional performance measures such as fuel consumption, air quality, and operating cost.

Conceptual Differences

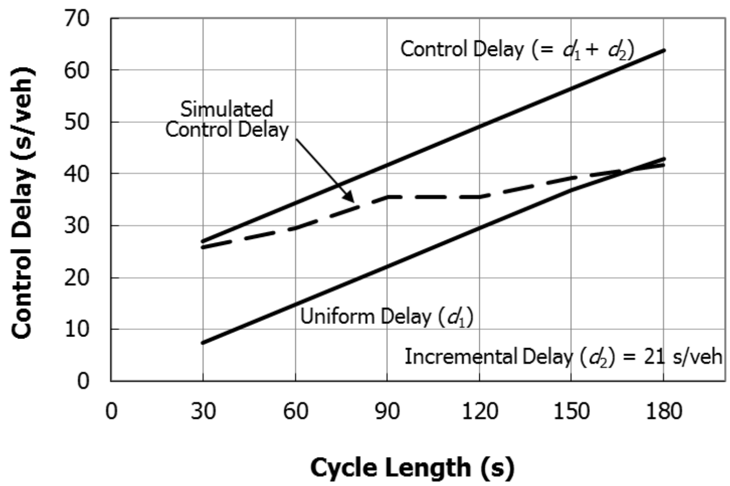
Conceptual differences in modeling approach may preclude the direct comparison of performance measures from the motorized vehicle methodology with those from alternative tools. The treatment of random arrivals is a case in point. There is a common misconception among analysts that alternative tools treat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

A simple case is used to demonstrate the different ways alternative tools model random arrivals. Consider an isolated intersection with a two-phase sequence. The subject intersection approach serves only a through movement;

there are no turning movements from upstream intersections or driveways. The only parameter that is allowed to vary in this example is the cycle length (all other variables are held constant).

The results of this experiment are shown in Exhibit 19-42. The two solid lines represent delay estimates obtained from the motorized vehicle methodology. Uniform delay is shown to increase linearly with cycle length. Incremental delay is constant with respect to cycle length because the volume-to-capacity ratio is constant. As a result, control delay (the sum of the uniform delay and incremental delay) is also shown to increase linearly with cycle length.

Exhibit 19-42
Effect of Cycle Length on Delay



The dashed line in Exhibit 19-42 represents the control delay estimate obtained from a simulation-based analysis tool. The simulation-based tool shows close agreement with the motorized vehicle methodology for short cycles, but it deviates for longer cycles. There are likely to be explainable reasons for this difference; however, the point is that such differences are likely to exist among tools. The analyst should understand the underlying modeling assumptions and limitations inherent in any tool (including the motorized vehicle methodology) when it is used. Moreover, the analyst should fully understand the definition of any performance measure used so as to interpret the results and observed trends properly.

Alternative Tool Application Guidance

Development of HCM-Compatible Performance Measures

The motorized vehicle methodology is used to predict control delay, which is defined as the excess travel time caused by the action of the control device (in this case, the signal). Simulation-based analysis tools often use a definition of delay that differs from that used in the motorized vehicle methodology, especially for movements that are oversaturated at some point during the analysis. Therefore, some care must be taken in the determination of LOS when simulation-based delay estimates are used. Delay comparison among different tools is discussed in more detail in Chapter 7.

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

Adjustment of Parameters

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

Sample Calculations Illustrating Alternative Tool Applications

Chapter 31, Signalized Intersections: Supplemental, includes examples to illustrate the use of simulation tools to address the stated limitations of this chapter. Specifically, these examples address the following conditions: left-turn storage-bay overflow, RTOR operation, short through lanes, and closely spaced intersections.

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Some of these references can be found in the Technical Reference Library in Volume 4.

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Enhancing Pedestrian Volume Estimation and Developing HCM Pedestrian Methodologies for Safe and Sustainable Communities

Chapter 20: Two-Way STOP-Controlled Intersections

Version 6.1 Final Draft

Prepared for:
NCHRP Project 17-87

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This chapter contains revisions to the pedestrian delay and LOS methodologies for TWSC intersections presented in Section 5, along with a small revision in the introduction, Section 1. These revisions were developed through NCHRP Project 17-87.

CHAPTER 20
TWO-WAY STOP-CONTROLLED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Two-way STOP-controlled (TWSC) intersections are common in the United States. One typical configuration is a four-leg intersection in which one street—the *major street*—is uncontrolled, and the other street—the *minor street*—is controlled by STOP signs. The other typical configuration is a three-leg intersection in which the single minor-street approach (i.e., the stem of the T configuration) is controlled by a STOP sign. Minor-street approaches can be public streets or private driveways. This chapter presents concepts and procedures for analyzing these types of intersections. Chapter 9 provides a glossary and list of symbols, including those used for TWSC intersections.

Three-leg intersections in which the stem of the T is controlled by a STOP sign are considered a standard type of TWSC intersection.

Capacity analysis of TWSC intersections requires a clear description and understanding of the interaction between travelers on the minor (i.e., STOP-controlled) approach with travelers on the major street. Both analytical and regression models have been developed to describe this interaction. Procedures described in this chapter rely primarily on field measurements of TWSC performance in the United States (*1*) that have been applied to a gap acceptance model developed and refined in Germany (*2*).

CHAPTER ORGANIZATION

This chapter is organized into the following sections:

- Section 1 (this section) introduces the chapter.
- Section 2 describes the basic concepts of the TWSC procedure. Most notably, the concept of gap acceptance—which is the basis of TWSC intersection operations—is described. Performance measures and level-of-service (LOS) criteria are also discussed.
- Section 3 provides the details of the TWSC intersection analysis procedure for the motorized vehicle mode, including required input data and detailed computational steps.
- Section 4 extends the motorized vehicle mode procedure to account for the effects of pedestrians on capacity.
- Section 5 presents a procedure for analyzing pedestrian operations at TWSC intersections and uncontrolled midblock crossings, including required data and computational steps.
- Section 6 qualitatively discusses bicycle operations at a TWSC intersection and directs the reader to related research.
- Section 7 describes example problems included in Volume 4, suggests applications for alternative tools, and provides guidance on interpreting analysis results.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 4, Traffic Operations and Capacity Concepts, introduces concepts of traffic flow and capacity that apply to TWSC intersections, including peak hour factor, gap acceptance, and control delay.
- Chapter 5, Quality and Level-of-Service Concepts, provides an overview of the LOS concept used throughout the HCM.
- Chapter 32, STOP-Controlled Intersections: Supplemental, provides example problems demonstrating the TWSC methodology.
- Case Study 1, U.S. 95 Corridor, and Case Study 5, Museum Road, in the *HCM Applications Guide* in Volume 4, demonstrate how this chapter's methods can be applied to the evaluation of an actual TWSC intersection.
- Section M, STOP-Controlled Intersections, in the *Planning and Preliminary Engineering Applications Guide to the HCM* in Volume 4, provides guidance on analyzing TWSC intersections in the context of a planning study.

2. CONCEPTS

TWSC intersections are unsignalized intersections at which drivers on the major street have priority over drivers on the minor-street approaches. Minor-street drivers must stop before entering the intersection. Left-turning drivers from the major street must yield to oncoming major-street through or right-turning traffic, but they are not required to stop in the absence of oncoming traffic.

The methodologies presented rely on the required input data listed in Section 3 to compute the *potential capacity* of each minor movement, which is ultimately adjusted, if appropriate, to compute a *movement capacity* for each movement. The movement capacity can be used to estimate the control delay by movement, by approach, and for the intersection as a whole. Queue lengths can also be estimated once movement capacities are determined.

The capacity of the controlled legs is based primarily on three factors: the distribution of gaps in the major stream, driver judgment in selecting the gaps, and the follow-up headways required by each driver in a queue.

At TWSC intersections, drivers on the STOP-controlled approaches are required to select gaps in the major-street flow in order to execute crossing or turning maneuvers. In the presence of a queue, each driver on the controlled approach must also spend time moving to the front-of-queue position and prepare to evaluate gaps in the major-street flow. Thus, the capacity of the controlled legs is based primarily on three factors: the distribution of gaps in the major-street traffic stream, driver judgment in selecting gaps through which to execute the desired maneuvers, and the follow-up headways required by each driver in a queue.

When traffic signals are present on the major street upstream of the subject intersection, flows may not be random but will likely have some platoon structure.

The basic capacity model assumes gaps in the conflicting movements are randomly distributed. When traffic signals are present on the major street upstream of the subject intersection, flows may not be random but will likely have some platoon structure.

For the analysis of the motorized vehicle mode, the methodology addresses special circumstances that may exist at TWSC intersections, including the following:

- Two-stage gap acceptance,
- Approaches with shared lanes,
- The presence of upstream traffic signals, and
- Flared approaches for minor-street right-turning vehicles.

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The intersection boundaries for a TWSC intersection analysis are assumed to be those of an isolated intersection (i.e., not affected by upstream or downstream intersections), with the exception of TWSC intersections that may be affected by vehicle platoons from upstream signals. This chapter presents methodologies to assess TWSC intersections for both pedestrians and motor vehicles. A discussion of how the procedures for motor vehicles could potentially apply to an analysis of bicycle movements is also provided.

GAP ACCEPTANCE THEORY

Gap acceptance models begin with the recognition that TWSC intersections give no positive indication or control to the driver on the minor street as to when it is appropriate to leave the stop line and enter the major street. The driver must determine when a gap on the major street is large enough to permit entry and when to enter on the basis of the relative priority of the competing movements. This decision-making process has been formalized analytically into what is commonly known as gap acceptance theory. Gap acceptance theory includes three basic elements: the size and distribution (availability) of gaps on the major street, the usefulness of these gaps to the minor-street drivers, and the relative priority of the various movements at the intersection.

Availability of Gaps

The first element in gap acceptance theory is the proportion of gaps of a particular size on the major street offered to the driver entering from a minor movement, as well as the pattern of vehicle arrival times. The distribution of gaps between the vehicles in the different streams has a major effect on the performance of the intersection.

Usefulness of Gaps

The second element is the extent to which drivers find gaps of a particular size useful when they attempt to enter the intersection. It is generally assumed in gap acceptance theory that drivers are both consistent and homogeneous. This assumption is not entirely correct. Studies have demonstrated not only that drivers have different gap acceptance thresholds but that the gap acceptance threshold of an individual driver often changes over time (*3*). In this manual, the critical headways and follow-up headways are considered representative of a statistical average of the driver population in the United States.

Relative Priority of Various Movements at the Intersection

The third element in gap acceptance theory concerns the ranking of each movement in a priority hierarchy. Typically, gap acceptance processes assume drivers on the major street are unaffected by the minor movements. If this assumption is not the case at a given intersection, the gap acceptance process has to be modified.

In using the TWSC intersection methodology, the priority of right-of-way given to each movement must be identified. Some movements have absolute priority; other movements must yield to higher-order movements. Movements can be categorized by right-of-way priority as follows:

Pedestrian movements crossing the major street are assumed to be Rank 2 for the automobile analysis procedure. The effect of Rank 1 vehicles yielding to pedestrians is included in the pedestrian analysis procedure.

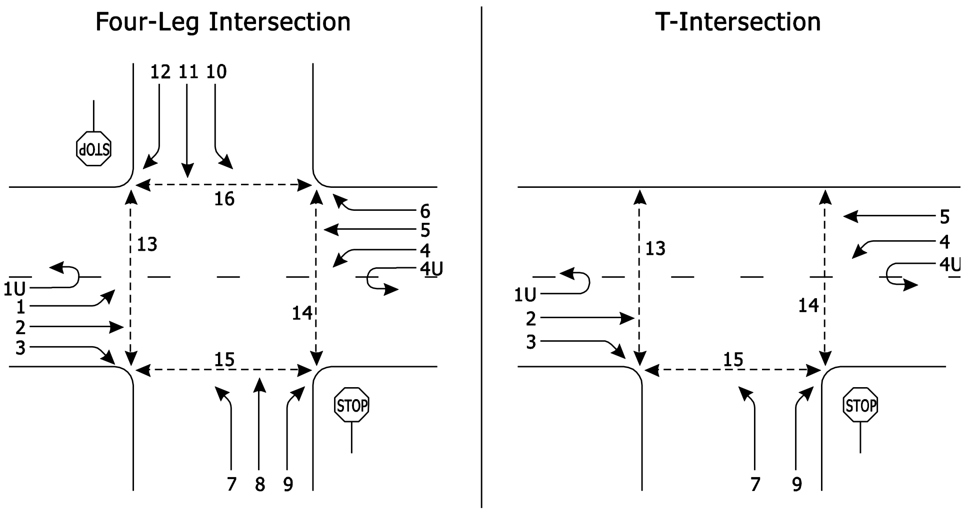
- Movements of Rank 1 include through traffic on the major street, right-turning traffic from the major street, and pedestrian movements crossing the minor street.
- Movements of Rank 2 (subordinate to Rank 1) include left-turning and U-turning traffic from the major street, right-turning traffic onto the major street, and pedestrian movements crossing the major street (assumed for this procedure).

- Movements of Rank 3 (subordinate to Ranks 1 and 2) include through traffic on the minor street (in the case of a four-leg intersection) and left-turning traffic from the minor street (in the case of a T-intersection).
- Movements of Rank 4 (subordinate to all others) include left-turning traffic from the minor street. Rank 4 movements occur only at four-leg intersections.

The minor-street left-turn movement is assigned Rank 3 priority at a T-intersection and Rank 4 priority at a four-leg intersection.

Exhibit 20-1 shows the assumed numbering of movements at both T- and four-leg intersections.

Exhibit 20-1
Vehicular and Pedestrian Movements at a TWSC Intersection



Solid arrows indicate vehicular movements; dashed arrows indicate pedestrian movements.

As an example of the application of right-of-way priority, assume the situation of a left-turning vehicle on the major street and a through vehicle from the minor street waiting to cross the major traffic stream. The first available gap of acceptable size would be taken by the left-turning vehicle. The minor-street through vehicle must wait for the second available gap. In aggregate terms, a large number of such left-turning vehicles could use up so many of the available gaps that minor-street through vehicles would be severely impeded or unable to make safe crossing movements.

Critical Headway and Follow-Up Headway

Critical headway defined.

Critical headway t_c is defined as the minimum time interval in the major-street traffic stream that allows intersection entry for one minor-street vehicle (*4*). Thus, the driver's critical headway is the minimum headway that would be acceptable. A particular driver would reject headways less than the critical headway and would accept headways greater than or equal to the critical headway. Critical headway can be estimated on the basis of observations of the largest rejected and smallest accepted headway for a given intersection.

Follow-up headway defined.

The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street, is called the *follow-up headway* t_f. Thus, t_f is the headway that defines the saturation flow rate for the approach if there were no conflicting vehicles on movements of higher rank.

LOS is not defined for the major-street approaches or for the overall intersection, as major-street through vehicles are assumed to experience no delay.

LEVEL-OF-SERVICE CRITERIA

LOS for a TWSC intersection is determined by the computed or measured control delay. For motor vehicles, LOS is determined for each minor-street movement (or shared movement), as well as the major-street left turns, by using the criteria given in Exhibit 20-2. LOS is not defined for the intersection as a whole or for major-street approaches for three primary reasons: (*a*) major-street through vehicles are assumed to experience zero delay; (*b*) the disproportionate number of major-street through vehicles at a typical TWSC intersection skews the weighted average of all movements, resulting in a very low overall average delay for all vehicles; and (*c*) the resulting low delay can mask LOS deficiencies for minor movements. As Exhibit 20-2 notes, LOS F is assigned to a movement if its volume-to-capacity ratio exceeds 1.0, regardless of the control delay.

The LOS criteria for TWSC intersections differ somewhat from the criteria used in Chapter 19 for signalized intersections, primarily because user perceptions differ among transportation facility types. The expectation is that a signalized intersection is designed to carry higher traffic volumes and will present greater delay than an unsignalized intersection. Unsignalized intersections are also associated with more uncertainty for users, as delays are less predictable than they are at signals.

Exhibit 20-2
LOS Criteria: Motorized Vehicle Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio $v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: The LOS criteria apply to each lane on a given approach and to each approach on the minor street. LOS is not calculated for major-street approaches or for the intersection as a whole.

Pedestrian LOS at TWSC intersections is defined for pedestrians crossing a traffic stream not controlled by a STOP sign; it also applies to midblock pedestrian crossings. Exhibit 20-3 gives LOS criteria for pedestrians, using a perception-based rating system considering the average proportions of pedestrians who would rate their crossing experience as "dissatisfied" or worse.

Exhibit 20-3
LOS Criteria: Pedestrian Mode

LOS	Condition	Comments
A	$P_D < 0.05$	Nearly all pedestrians would be satisfied
B	$0.05 \leq P_D < 0.15$	At least 85% of pedestrians would be satisfied
C	$0.15 \leq P_D < 0.25$	Fewer than one-quarter of pedestrians would be dissatisfied
D	$0.25 \leq P_D < 0.33$	Fewer than one-third of pedestrians would be dissatisfied
E	$0.33 \leq P_D < 0.50$	Fewer than one-half of pedestrians would be dissatisfied
F	$P_D \geq 0.50$	The majority of pedestrians would be dissatisfied

Note: P_D = proportion of pedestrians giving a "dissatisfied" rating or worse.

LOS F for pedestrians occurs when many pedestrians experience delay crossing the street, due to a lack of gaps in traffic, failure of motorists to yield, or both. LOS A and B occur when some combination of the following exists: pedestrian safety countermeasures exist at the crossing, traffic volumes are relatively low, and motorists frequently yield to pedestrians.

3. MOTORIZED VEHICLE CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

The version of the TWSC intersection analysis procedure presented in this section is primarily based on studies conducted by National Cooperative Highway Research Program Project 3-46 (*1*).

Spatial and Temporal Limits

This methodology assumes the TWSC intersection under investigation is isolated, with the exception of a TWSC intersection that may be affected by vehicle platoons from upstream signals. When interaction effects (e.g., queue spillback, demand starvation) are likely between the subject TWSC intersection and other intersections, the use of alternative tools may result in a more accurate analysis. Analysis boundaries may also include different demand scenarios related to the time of day or to different development scenarios that produce various demand flow rates.

The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This method produces the following performance measures:

- Volume-to-capacity ratio,
- Control delay,
- LOS based on control delay, and
- 95th percentile queue length.

Limitations of the Methodology

The methodologies in this chapter apply to TWSC intersections with up to three through lanes (either shared or exclusive) on the major-street approaches and up to three lanes on the minor-street approaches (with no more than one exclusive lane for each movement on the minor-street approach). Effects from other intersections are accounted for only in situations in which a TWSC intersection is located on an urban street segment between coordinated signalized intersections. In this situation, the intersection can be analyzed by using the procedures in Chapter 18, Urban Street Segments. The methodologies do not apply to TWSC intersections with more than four approaches or more than one STOP-controlled approach on each side of the major street.

The methodologies do not include a detailed method for estimating delay at YIELD-controlled intersections; however, with appropriate changes in the values of key parameters (e.g., critical headway and follow-up headway), the analyst could apply the TWSC method to YIELD-controlled intersections.

With appropriate changes in the values of critical headway and follow-up headway, the analyst could apply the TWSC method to YIELD-controlled intersections.

All the methods are for steady state conditions (i.e., the demand and capacity conditions are constant during the analysis period); the methods are not designed to evaluate how fast or how often the facility transitions from one demand or capacity state to another. Analysts interested in that kind of information should consider applying alternative tools, as discussed below.

Alternative Tool Considerations

Strengths of the HCM Procedure

This chapter offers a set of comprehensive procedures for analyzing the performance of an intersection under two-way STOP control. Simulation-based tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with the roadway and the control system, but for most purposes the HCM procedure produces an acceptable approximation.

The HCM procedure offers the advantage of a deterministic evaluation of a TWSC intersection, the results of which have been accepted by a broad consensus of international experts. The HCM procedure also considers advanced concepts such as two-stage gap acceptance and flared approaches based on empirical evidence of their effects.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

The identified limitations for this chapter are shown in Exhibit 20-4, along with the potential for improved treatment by alternative tools.

Exhibit 20-4
Limitations of the HCM TWSC Intersection Motorized Vehicle Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Effects of upstream intersections	Simulation tools can include an unsignalized intersection explicitly within a signalized arterial or network.
YIELD-controlled intersection operations	Treated explicitly by some tools. Can be approximated by varying the gap acceptance parameters.
Non-steady-state conditions for demand and capacity	Most alternative tools provide for multiperiod variation of demand and, in some cases, capacity.
Atypical intersection configurations, such as more than four legs or STOP control on all but one leg	Some alternative tools can be customized to model the unique configuration of these types of intersections.

The most common application of alternative tools for TWSC analysis involves an unsignalized intersection located between coordinated signalized intersections.

Most analyses for isolated unsignalized intersections are intended to determine whether TWSC is a viable control alternative. Analyses of this type are handled adequately by the procedures described in this chapter. The most common application of alternative tools for TWSC analysis involves an unsignalized intersection located between coordinated signalized intersections. Most intersections between coordinated signalized intersections operate under TWSC. These intersections tend to be ignored in the analysis of the system because their effect on the system operation is minimal. Occasionally, it is necessary to examine a TWSC intersection as a part of the arterial system. Although the procedures in this chapter provide a method for approximating the operation of a TWSC intersection with an upstream signal, the operation of such an intersection is arguably best handled by including it in a complete simulation of the full arterial system. For example, queue backup from a downstream signal that blocks entry from the cross street for a portion of the cycle is not treated explicitly by the procedures contained in this chapter.

Another potential application for alternative tools is modeling intersections with more than four legs or with control configurations other than the typical priority control, such as STOP control on all but one leg. The operation of these types of intersections has not been adequately researched, and no analytical method has been developed to model their operation. It may be possible to use an alternative tool to model these configurations provided the priorities between movements can be customized to match field operations.

Development of HCM-Compatible Performance Measures Using Alternative Tools

Control delay, the performance measure that determines LOS for TWSC intersections, is defined as that portion of the delay caused by a control device—in this case, a STOP sign. Most simulation tools do not produce explicit estimates of control delay.

The best way to determine control delay at a STOP sign from simulation is to perform simulation runs with and without the control device in place. The segment delays reported with no control represent the delays due to geometrics and interaction between vehicles. The additional delay reported in the run with the control device in place is, by definition, the control delay.

Chapter 7, Interpreting HCM and Alternative Tool Results, discusses performance measures from various tools in more detail, and Chapter 36, Concepts: Supplemental, provides recommendations on how individual vehicle trajectories should be interpreted to produce specific performance measures. Of particular interest to TWSC operation is the definition of a queued state and the development of queue delay from that definition. For alternative tools that conform to the queue delay definitions and computations presented in this manual, queue delay will provide the best estimate of control delay for TWSC intersections. Delay and LOS should not be estimated by using alternative tools that do not conform to these definitions and computations.

Delay and LOS should be estimated only by using alternative tools that conform to the definitions and computations of queue delay presented in this manual.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Deterministic tools and simulation tools both model TWSC operations as a gap acceptance process that follows the rules of the road to determine the right-of-way hierarchy. To this extent, both types of tools use the same conceptual framework. Deterministic tools such as the HCM base their estimates of capacity and delay on expected values computed from analytical formulations that have been mathematically derived. Simulation tools, in contrast, take a more microscopic view, treating each vehicle as an independent object that is subject to the rules of the road as well as interaction with other vehicles. Differences in the treatment of randomness also exist, as explained in the guidance provided in Chapter 19, Signalized Intersections.

When the opposing movement volumes are very high, there is minimal opportunity for the STOP-controlled movements to accept gaps, and these movements often have little or no capacity. Simulation tends to produce slightly higher capacities under these conditions because of a tool-specific overriding logic that limits the amount of time any driver is willing to wait for a gap.

In general, the simulation results for a specific TWSC intersection problem should be close to the results obtained from the procedures in this chapter. Some differences may, however, be expected among the analysis tools.

Adjustment of Simulation Parameters to the HCM Parameters

Critical headways and follow-up headways are common to both deterministic and simulation models. It is therefore desirable that similar values be used for these parameters.

Sample Calculations Illustrating Alternative Tool Applications

An example of the most common application for TWSC simulation, unsignalized intersections within a signalized arterial system, is presented in Chapter 29, Urban Street Facilities: Supplemental. An additional example involving blockage of a cross-street approach with STOP control by a queue from a nearby diamond interchange is presented in Chapter 34, Interchange Ramp Terminals: Supplemental.

REQUIRED INPUT DATA AND SOURCES

Exhibit 20-5 lists the information necessary to apply the motorized vehicle methodology and suggests potential sources for obtaining these data. It also suggests default values for use when intersection-specific information is not available.

Exhibit 20-5
Required Input Data, Potential Data Sources, and Default Values for TWSC Motorized Vehicle Analysis

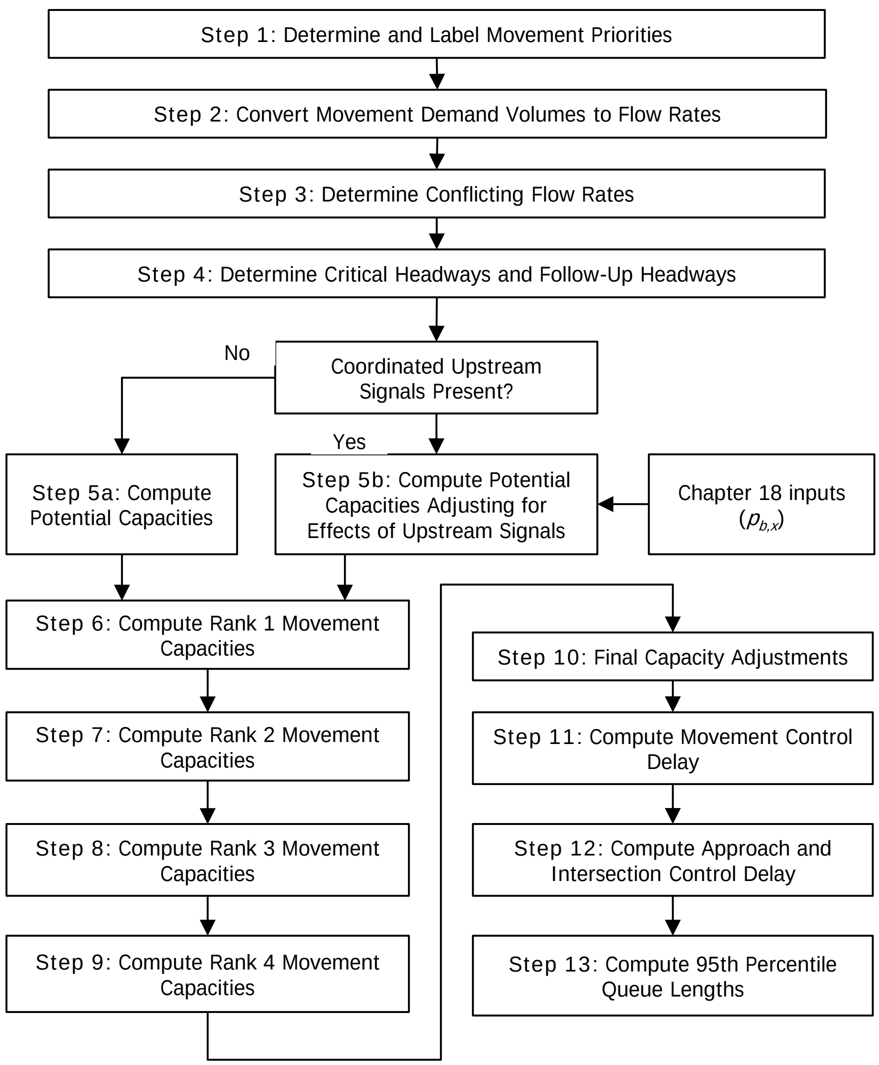
Required Data and Units	Potential Data Source(s)	Suggested Default Value
Geometric Data		
Number and configuration of lanes of each approach	Design plans, road inventory	Must be provided
Approach grades	Design plans, road inventory	0%
Special geometric factors such as • Unique channelization aspects • Existence of a two-way left-turn lane or raised or striped median storage (or both) • Existence of flared approaches on the minor street • Existence of upstream signals	Design plans, road inventory	Must be provided
Demand Data		
Hourly turning-movement demand volume (veh/h) AND peak hour factor OR Hourly turning-movement demand flow rate (veh/h)	Field data, modeling	Must be provided
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor (decimal)	Field data	0.92
Heavy-vehicle percentage (%)	Field data	3%
Saturation flow rate for major-street through movement (for analysis of shared or short major-street, left-turn lanes)	Field data	1,800 veh/h
Saturation flow rate for major-street, right-turn movement (for analysis of shared or short major-street, left-turn lanes)	Field data	1,500 veh/h

A comprehensive presentation of potential default values for interrupted flow facilities is provided elsewhere (*5*), with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of peak hour factor and percentage of heavy vehicles. Recommendations are based on geographic region, population, and time of day. All general default values for interrupted-flow facilities may be applied to the analysis of TWSC intersections in the absence of field data or projected conditions.

COMPUTATIONAL STEPS

The TWSC intersection methodology for the motorized vehicle mode is applied through a series of steps that require input data related to movement flow information and geometric conditions, prioritization of movements, computation of potential capacities, incorporation of adjustments to compute movement capacities, and estimation of control delays and queue lengths. These steps are illustrated in Exhibit 20-6.

Exhibit 20-6
TWSC Intersection Methodology



Step 1: Determine and Label Movement Priorities

The priority for each movement at a TWSC intersection must be identified to designate the appropriate rank of each movement for future steps in the analysis process. This step's process also identifies the sequence in which the analyst will complete the capacity computations. Because the methodology is based on prioritized use of gaps by vehicles at a TWSC intersection, the subsequent computations must be made in a precise order. The computational sequence is the same as the priority of gap use, and movements are considered in the following order:

1. Left turns from the major street,
2. Right turns from the minor street,
3. U-turns from the major street,
4. Through movements from the minor street, and
5. Left turns from the minor street.

Step 2: Convert Movement Demand Volumes to Flow Rates

For analysis of existing conditions when the peak 15-min period can be measured in the field, the volumes for the peak 15-min period are converted to a peak 15-min demand flow rate by multiplying the peak 15-min volumes by four.

For analysis of projected conditions or when 15-min data are not available, hourly demand volumes for each movement are converted to peak 15-min demand flow rates in vehicles per hour, as shown in Equation 20-1, through use of the peak hour factor for the intersection.

Equation 20-1

$$v_i = \frac{V_i}{PHF}$$

where

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

V_i = demand volume for movement i (veh/h), and

PHF = peak hour factor for the intersection.

If PHF is used, a single intersectionwide PHF should be used rather than movement-specific or approach-specific PHFs. If individual approaches or movements peak at different times, a series of 15-min analysis periods that encompasses the peaking should be considered.

The use of a peak 15-min traffic count multiplied by four is preferred for existing conditions when traffic counts are available. The use of a 1-h demand volume divided by a peak hour factor is preferred with projected volumes or with projected volumes that have been added to current volumes.

If peak hour factors are used, a single peak hour factor for the entire intersection is generally preferred to decrease the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, but in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. If individual approaches or movements are known to have substantially different peaking characteristics or peak during different 15-min periods within the hour, a series of 15-min analysis periods that encompasses the peaking should be considered instead of a single analysis period using a single peak hour factor for the intersection.

Step 3: Determine Conflicting Flow Rates

Each movement at a TWSC intersection faces a different set of conflicts that is directly related to the nature of the subject movement. The following subsections describe the set of conflicts facing each minor movement (Rank 2 through Rank 4) at a TWSC intersection. The exhibits and equations illustrate the computation of the parameter $v_{c,x}$, the conflicting flow rate for movement x—that is, the total flow rate (in vehicles per hour) that conflicts with movement x.

Pedestrians may also conflict with vehicular movements. Pedestrian flow rates (defined as v_x, with x noting the leg of the intersection being crossed) should be included as part of the conflicting flow rates. Pedestrian flows are included because they define the beginning or ending of a gap that may be used by a minor-street movement. Although this method recognizes some peculiarities associated with pedestrian movements, it takes a uniform approach to vehicular and pedestrian movements.

Major-Street Left-Turn Movements: Rank 2, Movements 1 and 4

Exhibit 20-7 illustrates the conflicting movements and Equation 20-2 and Equation 20-3 compute the conflicting flow rates encountered by major-street left-turning drivers. The left-turn movement from the major street conflicts with the total opposing through and right-turn flow, because the left-turning vehicles must cross the opposing through movement and be in conflict with the right-turning vehicles. The method does not differentiate between crossing and merging conflicts. Left-turning vehicles from the major street and the opposing right-turning vehicles from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

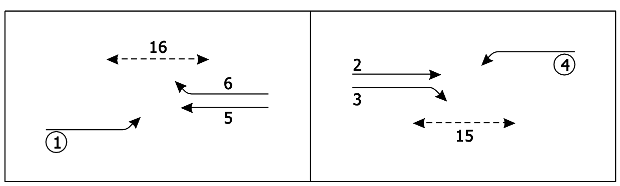


Exhibit 20-7
Illustration of Conflicting Movements for Major-Street Left-Turn Movements

$$v_{c,1} = v_5 + v_6 + v_{16}$$ (Equation 20-2)

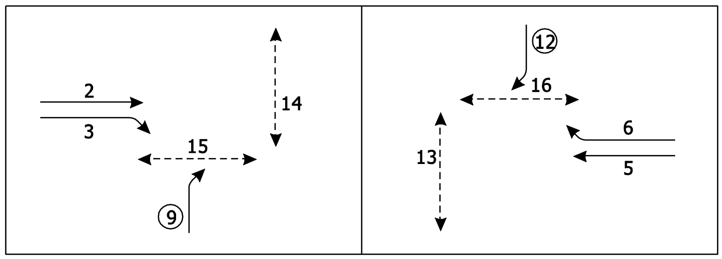
$$v_{c,4} = v_2 + v_3 + v_{15}$$ (Equation 20-3)

If the major-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, then the v_6 and v_3 terms in Equation 20-2 and Equation 20-3, respectively, may be assumed to be zero.

Minor-Street Right-Turn Movements: Rank 2, Movements 9 and 12

Exhibit 20-8 illustrates the conflicting movements encountered by minor-street right-turning drivers. The right-turn movement from the minor street is assumed to be in conflict with only a portion of the major-street through movement when more than one major-street lane is present. Also, one-half of each right-turn movement from the major street is considered to conflict with the minor-street right-turn movement, as some of these turns tend to inhibit the subject movement. Because right-turning vehicles from the minor street commonly merge into gaps in the right-hand lane of the stream into which they turn, they typically do not require a gap across all lanes of the conflicting stream (this situation may not be true for some trucks and vans with long wheelbases that encroach on more than one lane in making their turns). Furthermore, a gap in the overall major-street traffic could be used simultaneously by another vehicle, such as a major-street left-turning vehicle. Exhibit 20-8 does not include vehicles making major-street U-turns as conflicting vehicles. Although these conflicts may be observed in practice, they are not assumed to be conflicts in this methodology.

Exhibit 20-8
Illustration of Conflicting Movements for Minor-Street Right-Turn Movements



Equation 20-4 through Equation 20-9 compute the conflicting flow rates for minor-street right-turn movements entering a major street. If the major-street right turn has its own lane, the corresponding v_3 or v_6 term in these equations may be assumed to be zero. Users may supply different lane distributions for the v_2 and v_5 terms in the equations for four- and six-lane major streets when supported by field data.

Equation 20-4 and Equation 20-5 compute the conflicting flow rates for minor-street right-turn movements entering two-lane major streets:

Equation 20-4
$$v_{c,9} = v_2 + 0.5v_3 + v_{14} + v_{15}$$

Equation 20-5
$$v_{c,12} = v_5 + 0.5v_6 + v_{13} + v_{16}$$

Equation 20-6 and Equation 20-7 are used for four-lane major streets:

Equation 20-6
$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15}$$

Equation 20-7
$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{13} + v_{16}$$

Equation 20-8 and Equation 20-9 are used for six-lane major streets:

Equation 20-8
$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15}$$

Equation 20-9
$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{13} + v_{16}$$

Major-Street U-Turn Movements: Rank 2, Movements 1U and 4U

Exhibit 20-9 illustrates the conflicting movements encountered by major-street U-turning drivers. The U-turn movement from the major street conflicts with the total opposing through and right-turn flow, similar to the major-street left-turn movement. Research has found that the presence of minor-street right-turning vehicles significantly affects the capacity of major-street U-turns (*6*). The methodology accounts for this effect in the impedance calculation rather than in the calculation of conflicting flow. If a different priority order is desired (e.g., minor-street right turns yield to major-street U-turns), the analyst should adjust the computation procedure accordingly to replicate observed conditions.

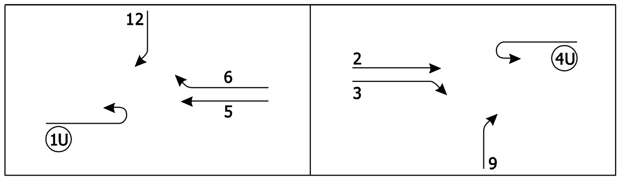


Exhibit 20-9
Illustration of Conflicting Movements for Major-Street U-Turn Movements

Equation 20-10 through Equation 20-13 compute the conflicting flow rates for major-street U-turns. No field data are available for U-turns on major streets with fewer than four lanes. If a major-street right turn has its own lane, the corresponding v_3 or v_6 term in these equations should be assumed to be zero.

Equation 20-10 and Equation 20-11 compute the conflicting flow rates for major-street U-turns when the major street has four lanes:

$$v_{c,1U} = v_5 + v_6$$

Equation 20-10

$$v_{c,4U} = v_2 + v_3$$

Equation 20-11

Equation 20-12 and Equation 20-13 compute the conflicting flow rates for major-street U-turns on six-lane major streets:

$$v_{c,1U} = 0.73v_5 + 0.73v_6$$

Equation 20-12

$$v_{c,4U} = 0.73v_2 + 0.73v_3$$

Equation 20-13

Minor-Street Pedestrian Movements: Rank 2, Movements 13 and 14

Minor-street pedestrian movements (those pedestrians crossing the major street) are in direct conflict with all vehicular movements on the major street except the right-turn and left-turn movements on the major street approaching from the far side of the intersection. The volume of minor-street pedestrians is an input parameter in the computation of the conflicting flow rates for all Rank 3 and Rank 4 movements.

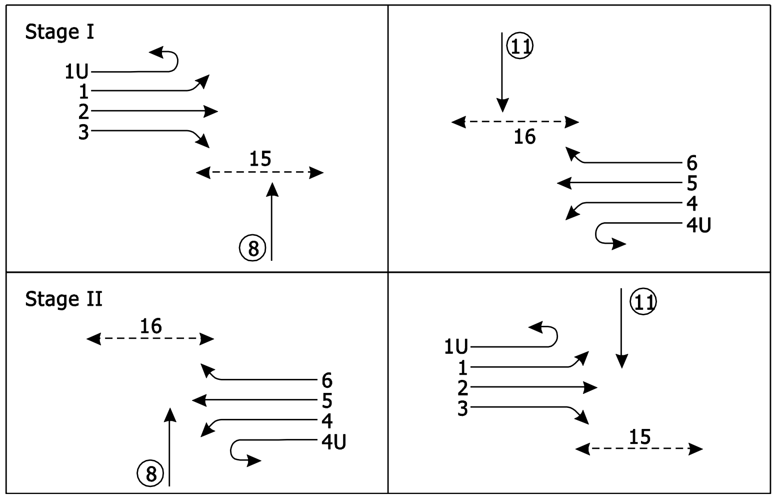
Minor-Street Through Movements: Rank 3, Movements 8 and 11

Minor-street through movements have a direct crossing or merging conflict with all movements on the major street except the right turn into the subject approach. Similar to the minor-street right-turn movement, one-half of each right-turn movement from the major street is considered to conflict with the minor-street through movement. In addition, field research (*1*) has shown that the effect of left-turning vehicles is approximately twice their actual number.

Drivers executing minor-street through movements may complete their maneuver in one or two stages. One-stage gap acceptance assumes no median refuge area is available for minor-street drivers to store in and that the minor-street drivers will evaluate gaps in both major-street directions simultaneously. Conversely, the two-stage gap acceptance scenario assumes a median refuge area is available for minor-street drivers. During Stage I, minor-street drivers evaluate major-street gaps in the nearside traffic stream (conflicting traffic from the left); during Stage II, minor-street drivers evaluate major-street gaps in the farside traffic stream (conflicting traffic from the right). For one-stage crossings, the conflicting flows for Stage I and Stage II are combined; for two-stage crossings, the conflicting flows are considered separately.

Exhibit 20-10 illustrates the conflicting movements encountered by minor-street through-movement drivers.

Exhibit 20-10
Illustration of Conflicting Movements for Minor-Street Through Movements



Equation 20-14 and Equation 20-15 compute the conflicting flows encountered by minor-street through-movement drivers during Stage I. If there is a right-turn lane on the major street, the corresponding v_3 or v_6 term in these equations may be assumed to be zero.

Equation 20-14
$$v_{c,\mathrm{I},8} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

Equation 20-15
$$v_{c,\mathrm{I},11} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$

Equation 20-16 and Equation 20-17 compute the conflicting flows encountered by minor-street through-movement drivers during Stage II. If the major-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, the corresponding v_3 or v_6 term in these equations may be assumed to be zero.

Equation 20-16
$$v_{c,\mathrm{II},8} = 2(v_4 + v_{4U}) + v_5 + v_6 + v_{16}$$

Equation 20-17
$$v_{c,\mathrm{II},11} = 2(v_1 + v_{1U}) + v_2 + v_3 + v_{15}$$

Minor-Street Left-Turn Movements: Rank 4, Movements 7 and 10

The left-turn movement from the minor street is the most difficult maneuver to execute at a TWSC intersection, and it faces the most complex set of conflicting movements, which include all major-street movements in addition to the opposing right-turn and through movements on the minor street. Only one-half the opposing right-turn and through-movement flow rate is included as conflicting flow rate because both movements are STOP-controlled, which diminishes their effect on left turns. The additional capacity impedance effects of the opposing right-turn and through-movement flow rates are taken into account elsewhere in the procedure.

Similar to minor-street through movements, minor-street left-turn movements may be completed in one or two stages. Exhibit 20-11 illustrates the conflicting movements encountered by minor-street left-turning drivers.

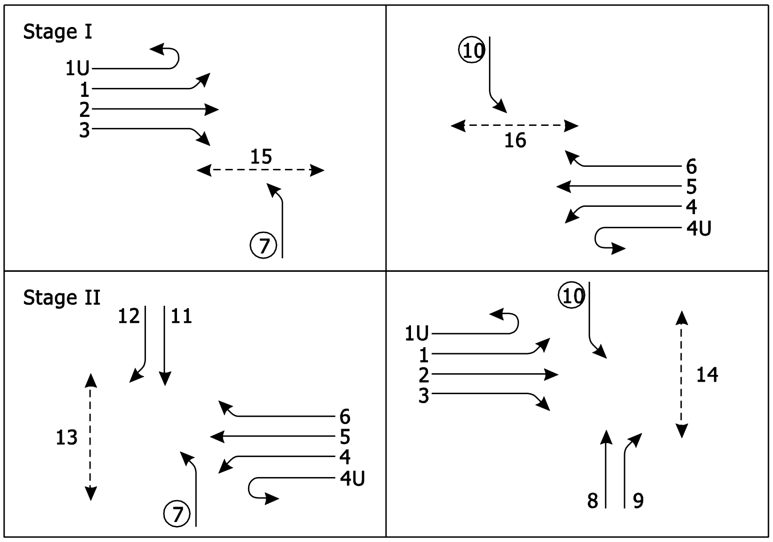


Exhibit 20-11
Illustration of Conflicting Movements for Minor-Street Left-Turn Movements

Equation 20-18 through Equation 20-23 compute the conflicting flow rates for minor-street left-turn movements entering a major street during Stage I. If a right-turn lane exists on the major street, the corresponding v_3 or v_6 term in these equations may be assumed to be zero.

During Stage I, Equation 20-18 and Equation 20-19 compute the conflicting flow rates for minor-street left-turn movements entering two-lane major streets:

$$v_{c,\mathrm{I},7} = 2v_1 + v_2 + 0.5v_3 + v_{15}$$ Equation 20-18

$$v_{c,\mathrm{I},10} = 2v_4 + v_5 + 0.5v_6 + v_{16}$$ Equation 20-19

Equation 20-20 and Equation 20-21 are used for four-lane major streets:

$$v_{c,\mathrm{I},7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$ Equation 20-20

$$v_{c,\mathrm{I},10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$ Equation 20-21

Equation 20-22 and Equation 20-23 are used for six-lane major streets:

$$v_{c,\mathrm{I},7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$ Equation 20-22

$$v_{c,\mathrm{I},10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$ Equation 20-23

Similarly, Equation 20-24 though Equation 20-29 compute the conflicting flow rates for minor-street left-turn movements entering a major street during Stage II. If the minor-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, the corresponding v_9 or v_{12} term in these equations may be assumed to be zero.

During Stage II, Equation 20-24 and Equation 20-25 compute the conflicting flow rates for minor-street left-turn movements entering two-lane major streets:

$$v_{c,\mathrm{II},7} = 2v_4 + v_5 + 0.5v_6 + 0.5v_{12} + 0.5v_{11} + v_{13}$$ Equation 20-24

$$v_{c,\mathrm{II},10} = 2v_1 + v_2 + 0.5v_3 + 0.5v_9 + 0.5v_8 + v_{14}$$ Equation 20-25

Equation 20-26 and Equation 20-27 are used for four-lane major streets:

$$v_{c,\mathrm{II},7} = 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$ Equation 20-26

$$v_{c,\mathrm{II},10} = 2(v_1 + v_{1U}) + 0.5v_2 + 0.5v_8 + v_{14}$$ Equation 20-27

Equation 20-28 and Equation 20-29 are used for six-lane major streets:

Equation 20-28

$$v_{c,\mathrm{II},7} = 2(v_4 + v_{4U}) + 0.4v_5 + 0.5v_{11} + v_{13}$$

Equation 20-29

$$v_{c,\mathrm{II},10} = 2(v_1 + v_{1U}) + 0.4v_2 + 0.5v_8 + v_{14}$$

Step 4: Determine Critical Headways and Follow-Up Headways

The critical headways $t_{c,x}$ and follow-up headways $t_{f,x}$ must be determined for the major-street left turns ($v_{c,1}$ and $v_{c,4}$), the minor-street right turns ($v_{c,9}$ and $v_{c,12}$), the major-street U-turns ($v_{c,1U}$ and $v_{c,4U}$), the minor-street through movements ($v_{c,8}$ and $v_{c,11}$), and the minor-street left turns ($v_{c,7}$ and $v_{c,10}$) as they occur at a TWSC intersection.

To compute the critical headways for each movement, the analyst begins with the base critical headway given in Exhibit 20-12 and makes movement-specific adjustments relating to the percentage of heavy vehicles, the grade encountered, and a three-leg versus four-leg intersection as shown in Equation 20-30.

Equation 20-30

$$t_{c,x} = t_{c,\mathrm{base}} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

where

$t_{c,x}$ = critical headway for movement x (s),

$t_{c,\mathrm{base}}$ = base critical headway from Exhibit 20-12 (s),

$t_{c,HV}$ = adjustment factor for heavy vehicles (1.0 for major streets with one lane in each direction; 2.0 for major streets with two or three lanes in each direction) (s),

P_{HV} = proportion of heavy vehicles for movement (expressed as a decimal; e.g., $P_{HV} = 0.02$ for 2% heavy vehicles),

$t_{c,G}$ = adjustment factor for grade for given movement (0.1 for Movements 9 and 12; 0.2 for Movements 7, 8, 10, and 11) (s),

G = percentage grade (expressed as an integer; e.g., $G = -2$ for a 2% downhill grade), and

$t_{3,LT}$ is applicable to Movements 7, 8, 10, and 11.

$t_{3,LT}$ = adjustment factor for intersection geometry (0.7 for minor-street left-turn movement at three-leg intersections; 0.0 otherwise) (s).

Exhibit 20-12
Base Critical Headways for TWSC Intersections

Vehicle Movement	Base Critical Headway, $t_{c,\mathrm{base}}$ (s) Two Lanes	Four Lanes	Six Lanes
Left turn from major street	4.1	4.1	5.3
U-turn from major street	NA	6.4 (wide)[a] 6.9 (narrow)[a]	5.6
Right turn from minor street	6.2	6.9	7.1
Through traffic on minor street	1 stage: 6.5 2 stage, Stage I: 5.5 2 stage, Stage II: 5.5	1 stage: 6.5 2 stage, Stage I: 5.5 2 stage, Stage II: 5.5	1 stage: 6.5[b] 2 stage, Stage I: 5.5[b] 2 stage, Stage II: 5.5[b]
Left turn from minor street	1 stage: 7.1 2 stage, Stage I: 6.1 2 stage, Stage II: 6.1	1 stage: 7.5 2 stage, Stage I: 6.5 2 stage, Stage II: 6.5	1 stage: 6.4 2 stage, Stage I: 7.3 2 stage, Stage II: 6.7

Notes: NA = not available.
[a] Narrow U-turns have a median nose width <21 ft; wide U-turns have a median nose width ≥21 ft.
[b] Use caution; values estimated.

The critical headway data for four- and six-lane sites account for the actual lane distribution of traffic flows measured at each site. For six-lane sites, minor-street left turns were commonly observed beginning their movement while apparently conflicting vehicles in the farside major-street through stream passed. The values for critical headway for minor-street through movements at six-lane streets are estimated, as the movement is not frequently observed in the field.

Similar to the computation of critical headways, the analyst begins the computation of follow-up headways with the base follow-up headways given in Exhibit 20-13. The analyst then makes movement-specific adjustments to the base follow-up headways with information gathered on heavy vehicles and the geometrics of the major street per the adjustment factors given in Equation 20-31.

Equation 20-31

$$t_{f,x} = t_{f,\text{base}} + t_{f,HV} P_{HV}$$

where

$t_{f,x}$ = follow-up headway for movement x (s),

$t_{f,\text{base}}$ = base follow-up headway from Exhibit 20-13 (s),

$t_{f,HV}$ = adjustment factor for heavy vehicles (0.9 for major streets with one lane in each direction; 1.0 for major streets with two or three lanes in each direction), and

P_{HV} = proportion of heavy vehicles for movement (expressed as a decimal; e.g., P_{HV} = 0.02 for 2% heavy vehicles).

Exhibit 20-13
Base Follow-Up Headways for TWSC Intersections

	Base Follow-Up Headway, $t_{f,\text{base}}$ (s)		
Vehicle Movement	Two Lanes	Four Lanes	Six Lanes
Left turn from major street	2.2	2.2	3.1
U-turn from major street	NA	2.5 (wide)[a] 3.1 (narrow)[a]	2.3
Right turn from minor street	3.3	3.3	3.9
Through traffic on minor street	4.0	4.0	4.0
Left turn from minor street	3.5	3.5	3.8

Notes: NA = not available.
[a] Narrow U-turns have a median nose width <21 ft; wide U-turns have a median nose width ≥21 ft.

Values from Exhibit 20-12 and Exhibit 20-13 are based on studies throughout the United States and are representative of a broad range of conditions. If smaller values for t_c and t_f are observed, capacity will be increased. If larger values for t_c and t_f are used, capacity will be decreased.

Step 5: Compute Potential Capacities

Step 5a: Potential Capacity Without Upstream Signal Effects

The potential capacity $c_{p,x}$ of a movement is computed according to the gap acceptance model provided in Equation 20-32 (*7*).

Equation 20-32

$$c_{p,x} = v_{c,x} \frac{e^{-v_{c,x} t_{c,x}/3{,}600}}{1 - e^{-v_{c,x} t_{f,x}/3{,}600}}$$

where

$c_{p,x}$ = potential capacity of movement x (veh/h),

$v_{c,x}$ = conflicting flow rate for movement x (veh/h),

$t_{c,x}$ = critical headway for minor movement x (s), and

$t_{f,x}$ = follow-up headway for minor movement x (s).

For two-stage Rank 3 or Rank 4 movements, the potential capacity is computed three times: $c_{p,x}$ assuming one-stage operation, $c_{p,\mathrm{I},x}$ for Stage I, and $c_{p,\mathrm{II},x}$ for Stage II. The conflicting flow definitions for each calculation are as provided in Step 4.

Step 5b: Potential Capacity with Upstream Signal Effects

To evaluate the impact of coordinated upstream signals, the urban street segments methodology (Chapter 17) is used to estimate the proportion of time that each Rank 2 or lower movement will be effectively blocked by a platoon. The proportion of time blocked is denoted by $p_{b,x}$, where x is the movement using the movement conventions provided in Exhibit 20-1.

With these values, the proportion of the analysis period that is blocked for each minor movement can be computed by using Exhibit 20-14.

Exhibit 20-14
Proportion of Analysis Period Blocked for Each Movement

	Proportion Blocked for Movement, $p_{b,x}$		
		Two-Stage Movements	
Movement(s) x	One-Stage Movements	Stage I	Stage II
1, 1U	$p_{b,1}$	NA	NA
4, 4U	$p_{b,4}$	NA	NA
7	$p_{b,7}$	$p_{b,4}$	$p_{b,1}$
8	$p_{b,8}$	$p_{b,4}$	$p_{b,1}$
9	$p_{b,9}$	NA	NA
10	$p_{b,10}$	$p_{b,1}$	$p_{b,4}$
11	$p_{b,11}$	$p_{b,1}$	$p_{b,4}$
12	$p_{b,12}$	NA	NA

Note: NA = not applicable.

The flow for the unblocked period (no platoons) is determined in this step. This flow becomes the conflicting flow for the subject movement and is used to compute the capacity for this movement. The minimum platooned flow rate $v_{c,min}$ is approximately 1,000N, where N is the number of through lanes per direction on the major street (*8*).

The conflicting flow for movement x during the unblocked period is given by Equation 20-33.

Equation 20-33

$$v_{c,u,x} = \begin{cases} \dfrac{v_{c,x} - 1.5 v_{c,min} p_{b,x}}{1 - p_{b,x}} & \text{if } v_{c,x} > 1.5 v_{c,min} p_{b,x} \\ 0 & \text{otherwise} \end{cases}$$

where

$v_{c,u,x}$ = conflicting flow for movement x during the unblocked period (veh/h);

$v_{c,x}$ = total conflicting flow for movement x as determined from Step 3 (veh/h);

$v_{c,min}$ = minimum platooned flow rate (veh/h), assumed to be 1,000N, where N is the number of through lanes per direction on the major street; and

$p_{b,x}$ = proportion of time the subject movement x is blocked by the major-street platoon, which is determined from Exhibit 20-14.

The potential capacity of the subject movement x, accounting for the effect of platooning, is given by Equation 20-34 and Equation 20-35.

$$c_{p,x} = (1 - p_{b,x})c_{r,x}$$ Equation 20-34

$$c_{r,x} = v_{c,u,x} \frac{e^{-v_{c,u,x}t_{c,x}/3,600}}{1 - e^{-v_{c,u,x}t_{f,x}/3,600}}$$ Equation 20-35

where

$c_{p,x}$ = potential capacity of movement x (veh/h),

$p_{b,x}$ = proportion of time that movement x is blocked by a platoon, and

$c_{r,x}$ = capacity of movement x assuming random flow during the unblocked period.

These equations use the same critical headway and follow-up headway inputs as a normal calculation, but they use only the conflicting flow during the unblocked period.

Steps 6–9: Compute Movement Capacities

For clarity, these steps assume pedestrian impedance effects can be neglected, and in many cases this assumption is reasonable. However, pedestrians can be accounted for in the analysis of the motorized vehicle mode by replacing these steps with those provided in Section 4, Extension to the Motorized Vehicle Methodology, which incorporate the effects of pedestrian impedance.

Step 6: Compute Rank 1 Movement Capacities

Rank 1 major-street movements are assumed to be unimpeded by any movements of lower rank. This rank also implies that major-street movements of Rank 1 are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays do occasionally occur, and they are accounted for by using adjustments provided later in this procedure.

Step 7: Compute Rank 2 Movement Capacities

Movements of Rank 2 (left turns and U-turns from the major street and right turns from the minor street) must yield to conflicting major-street through and right-turning vehicular movements of Rank 1. Minor-street right turns are assumed to yield to major-street U-turns, although sometimes the reverse occurs.

Step 7a: Movement Capacity for Major-Street Left-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 major-street left-turn movements (1 and 4) is equal to its potential capacity $c_{p,j}$, as shown in Equation 20-36.

$$c_{m,j} = c_{p,j}$$ Equation 20-36

Step 7b: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 minor-street right-turn movements (9 and 12) is equal to its potential capacity $c_{p,j}$, as shown in Equation 20-37.

$$c_{m,j} = c_{p,j}$$ Equation 20-37

Step 7c: Movement Capacity for Major-Street U-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 major-street U-turn movements (1U and 4U) is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. Field observations are mixed in terms of the degree to which major-street U-turn movements yield to minor-street right-turn movements and vice versa (*5*). It is assumed that the presence of minor-street right-turning vehicles will impede U-turning vehicles from accepting gaps in the major-street traffic stream; therefore, the capacity of the U-turn movement is affected by the probability that the minor-street right-turning traffic will operate in a queue-free state. The capacity adjustment factors are denoted by f_{1U} and f_{4U} for the major-street U-turn movements 1U and 4U, respectively, and are given by Equation 20-38 and Equation 20-39, respectively.

Equation 20-38

$$f_{1U} = p_{0,12} = 1 - \frac{v_{12}}{c_{m,12}}$$

Equation 20-39

$$f_{4U} = p_{0,9} = 1 - \frac{v_9}{c_{m,9}}$$

where

f_{1U}, f_{4U} = capacity adjustment factor for Rank 2 major-street U-turn movements 1 and 4, respectively;

$p_{0,j}$ = probability that conflicting Rank 2 minor-street right-turn movement j will operate in a queue-free state;

v_j = flow rate of movement j;

$c_{m,j}$ = capacity of movement j; and

j = 9 and 12 (minor-street right-turn movements of Rank 2).

The movement capacity for major-street U-turn movements is then computed with Equation 20-40.

Equation 20-40

In almost all cases, major-street left-turning vehicles share a lane with U-turning vehicles. If Rank 2 major-street U-turn movements are present to a significant degree, then Equation 20-41 should be used to compute the shared-lane capacity.

$$c_{m,jU} = c_{p,jU} \times f_{jU}$$

where

$c_{m,jU}$ = movement capacity for Movements 1U and 4U,

$c_{p,jU}$ = potential capacity for Movements 1U and 4U (from Step 5), and

f_{jU} = capacity adjustment factor for Movements 1U and 4U.

Because left-turn and U-turn movements are typically made from the same lane, their shared-lane capacity is computed with Equation 20-41.

Equation 20-41

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}}$$

where

c_{SH} = capacity of the shared lane (veh/h),

v_y = flow rate of the y movement in the subject shared lane (veh/h), and

$c_{m,y}$ = movement capacity of the y movement in the subject shared lane (veh/h).

Step 7d: Effect of Major-Street Shared Through and Left-Turn Lane

The probability that the major-street left-turning traffic will operate in a queue-free state is expressed by Equation 20-42.

$$p_{0,j} = 1 - \frac{v_j}{c_{m,j}}$$

Equation 20-42

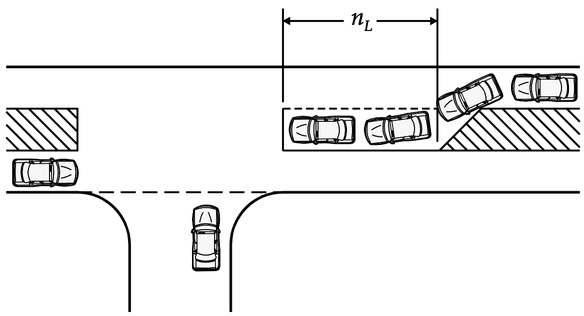
where j = 1 and 4 (major-street left-turn and U-turn movements of Rank 2, using shared volume and capacity as appropriate).

Use Equation 20-42 to compute the probability of a queue-free state for Rank 2 movements.

If major-street through and left-turn movements are shared, use Equation 20-43.

If, however, a shared left-turn lane or a short left-turn pocket is present on a major-street approach (as in Exhibit 20-15), the analyst accounts for this occurrence by computing the probability that there will be no queue in the major-street shared lane, $p^*_{0,j}$, according to Equation 20-43. This probability is then used by the analyst in lieu of $p_{0,j}$ from Equation 20-42.

Exhibit 20-15
Short Left-Turn Pocket on Major-Street Approach



The methodology implicitly assumes an exclusive lane is provided to all left-turning traffic from the major street. If a left-turn lane is not provided or the left-turn pocket is not long enough to accommodate all queuing left-turn and U-turn vehicles, major-street through (and possibly right-turning) traffic could be delayed by left-turning vehicles waiting for an acceptable gap in opposing major-street through traffic. To account for this occurrence, the factors $p^*_{0,1}$ and $p^*_{0,4}$ may be computed according to Equation 20-43 and Equation 20-44 as an indication of the probability there will be no queue in the respective major-street shared or short lanes (*9*).

$$p^*_{0,j} = 1 - \left(1 - p_{0,j}\right)\left[\sqrt[(n_L+1)]{1 + \frac{x_{i,1+2}^{(n_L+1)}}{1 - x_{i,1+2}}}\right]$$

Equation 20-43

$$x_{i,1+2} = \frac{v_{i1}}{s_{i1}} + \frac{v_{i2}}{s_{i2}}$$

Equation 20-44

where

$p_{0,j}$ = probability of queue-free state for movement j assuming an exclusive left-turn lane on the major street (per Equation 20-42);

$p^*_{0,j}$ = probability of queue-free state for movement j assuming a shared left-turn lane on the major street;

j = 1 and 4 (major-street left-turning vehicular movements);

When j = 1, i1 = 2 and i2 = 3; when j = 4, i1 = 5 and i2 = 6.

$i1$ = 2 and 5 (major-street through vehicular movements);

$i2$ = 3 and 6 (major-street right-turning vehicular movements);

$x_{i,1+2}$ = combined degree of saturation for the major-street through and right-turn movements;

s_{i1} = saturation flow rate for the major-street through movements (default assumed to be 1,800 veh/h; however, this parameter can be measured in the field);

s_{i2} = saturation flow rate for the major-street right-turn movements (default assumed to be 1,500 veh/h; however, this parameter can be measured in the field);

v_{i1} = major-street through-movement flow rate (veh/h);

v_{i2} = major-street right-turn flow rate (veh/h) (0 if an exclusive right-turn lane is provided); and

n_L = number of vehicles that can be stored in the left-turn pocket (see Exhibit 20-15).

For the special situation of shared lanes ($n_L = 0$), Equation 20-43 becomes Equation 20-45 as follows:

Equation 20-45

$$p_{0,j}^{*} = 1 - \frac{1 - p_{0,j}}{1 - x_{i,1+2}}$$

where all terms are as previously defined.

By using $p_{0,1}^{*}$ and $p_{0,4}^{*}$ in lieu of $p_{0,1}$ and $p_{0,4}$ (as computed by Equation 20-42), the potential for queues on a major street with shared or short left-turn lanes may be taken into account.

Step 8: Compute Rank 3 Movement Capacities

Rank 3 minor-street traffic movements (minor-street through movements at four-leg intersections and minor-street left turns at three-leg intersections) must yield to conflicting Rank 1 and Rank 2 movements. Not all gaps of acceptable length that pass through the intersection will normally be available for use by Rank 3 movements, because some of these gaps are likely to be used by Rank 2 movements.

If the Rank 3 movement is a two-stage movement, the movement capacity for the one-stage movement is computed as an input to the two-stage calculation.

Step 8a: Rank 3 Capacity for One-Stage Movements

For Rank 3 movements, the magnitude of vehicle impedance depends on the probability that major-street left-turning vehicles will be waiting for an acceptable gap at the same time as vehicles of Rank 3. A higher probability that this situation will occur means greater capacity-reducing effects of the major-street left-turning traffic on all Rank 3 movements.

The movement capacity $c_{m,k}$ for all Rank 3 movements is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor is denoted by f_k for all movements k and for all Rank 3 movements and is given by Equation 20-46.

Equation 20-46

$$f_k = \prod_j p_{0,j}$$

where Π indicates the product of a series of terms, and

$p_{0,j}$ = probability that conflicting Rank 2 movement j will operate in a queue-free state, and

k = Rank 3 movements.

The movement capacity $c_{m,k}$ for Rank 3 minor-street movements is computed with Equation 20-47.

$$c_{m,k} = c_{p,k} \times f_k$$

Equation 20-47

where $c_{p,k}$ is the potential capacity of Rank 3 minor-street movements, and f_k is the capacity adjustment factor that accounts for the impeding effects of higher-ranked movements computed according to Equation 20-46.

Step 8b: Rank 3 Capacity for Two-Stage Movements

If the Rank 3 movement is a two-stage movement, the procedure for computing the total movement capacity for the subject movement considering the two-stage gap acceptance process is as follows. An adjustment factor a and an intermediate variable y are computed with Equation 20-48 and Equation 20-49, respectively.

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} \quad \text{for } n_m > 0$$

Equation 20-48

$$y = \frac{c_{\mathrm{I}} - c_{m,x}}{c_{\mathrm{II}} - v_L - c_{m,x}}$$

Equation 20-49

where

n_m = number of vehicles that can be stored in the median;

c_{I} = movement capacity for the Stage I process (veh/h);

c_{II} = movement capacity for the Stage II process (veh/h);

v_L = major left-turn or U-turn flow rate, either $v_1 + v_{1U}$ or $v_4 + v_{4U}$ (veh/h); and

$c_{m,x}$ = capacity of subject movement, considering the total conflicting flow rate for both stages of a two-stage gap acceptance process (from Step 8a).

The terms c_{I}, c_{II}, and $c_{m,x}$ are capacities after being adjusted for upstream signals and impedance. Use $v_1 + v_{1U}$ when considering Movements 7 and 8 and $v_4 + v_{4U}$ when considering Movements 10 and 11.

The total capacity c_T for the subject movement, considering the two-stage gap acceptance process, is computed by using Equation 20-50 and Equation 20-51 and incorporating the adjustment factors derived from Equation 20-48 and Equation 20-49.

For $y \neq 1$:

$$c_T = \frac{a}{y^{n_m+1} - 1}\left[y(y^{n_m} - 1)(c_{\mathrm{II}} - v_L) + (y - 1)c_{m,x}\right]$$

Equation 20-50

For $y = 1$:

$$c_T = \frac{a}{n_m + 1}\left[n_m(c_{\mathrm{II}} - v_L) + c_{m,x}\right]$$

Equation 20-51

Step 9: Compute Rank 4 Movement Capacities

Rank 4 movements occur only at four-leg intersections. Rank 4 movements (i.e., only the minor-street left turns at a four-leg intersection) can be impeded by all higher-ranked movements (Ranks 1, 2, and 3).

Step 9a: Rank 4 Capacity for One-Stage Movements

The probability that higher-ranked traffic movements will operate in a queue-free state is central to determining their overall impeding effects on the minor-street left-turn movement. However, not all these probabilities are independent of each other. Specifically, queuing in the major-street left-turning movement affects the probability of a queue-free state in the minor-street crossing movement. Applying the simple product of these two probabilities will likely overestimate the impeding effects on the minor-street left-turning traffic.

Exhibit 20-16 can be used to adjust for the overestimate caused by the statistical dependence between queues in streams of Ranks 2 and 3. The mathematical representation of this curve is determined with Equation 20-52.

Equation 20-52

$$p' = 0.65p'' - \frac{p''}{p'' + 3} + 0.6\sqrt{p''}$$

where

p' = adjustment to the major-street left, minor-street through impedance factor;

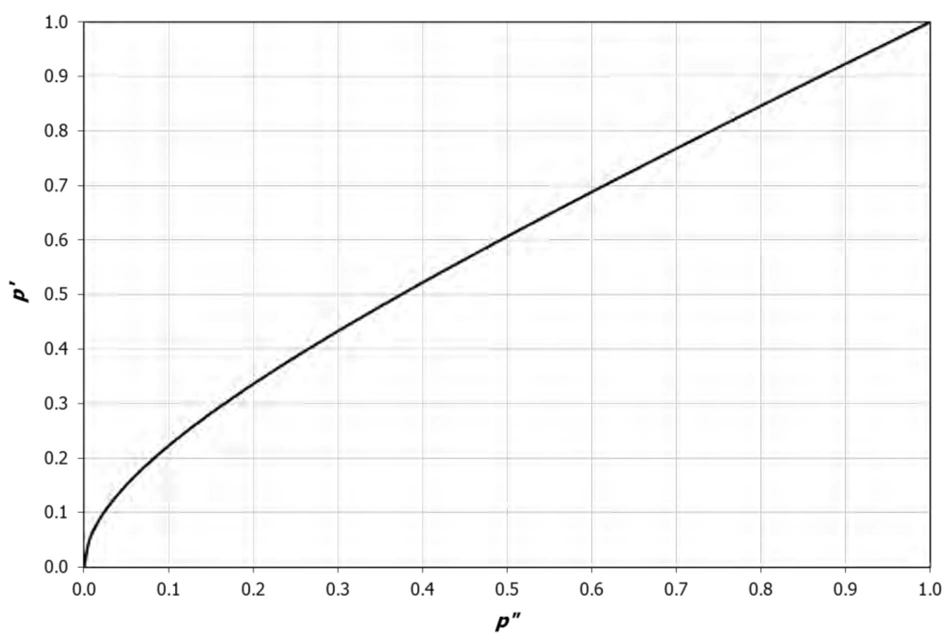
p'' = $(p_{0,j})(p_{0,k})$;

$p_{0,j}$ = probability of a queue-free state for the conflicting major-street left-turning traffic; and

$p_{0,k}$ = probability of a queue-free state for the conflicting minor-street crossing traffic.

When determining p' for Rank 4, Movement 7, in Equation 20-52, $p'' = (p_{0,1})(p_{0,4})(p_{0,11})$. Likewise, when determining p' for Rank 4, Movement 10, $p'' = (p_{0,1})(p_{0,4})(p_{0,8})$.

Exhibit 20-16
Adjustment to Impedance Factors for Major-Street Left-Turn Movement and Minor-Street Crossing Movement



The movement capacity $c_{m,l}$ for all Rank 4 movements is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor $f_{p,l}$ for the Rank 4 minor-street left-turn movement can be computed with Equation 20-53.

Equation 20-53

$$f_{p,l} = p' \times p_{0,j}$$

where

l = minor-street left-turn movement of Rank 4 (Movements 7 and 10 in Exhibit 20-1), and

j = conflicting Rank 2 minor-street right-turn movement (Movements 9 and 12 in Exhibit 20-1**).**

Finally, the movement capacity for the minor-street left-turn movements of Rank 4 is determined with Equation 20-54, where $f_{p,l}$ is the capacity adjustment factor that accounts for the impeding effects of higher-ranked movements.

Equation 20-54

$$c_{m,l} = c_{p,l} \times f_{p,l}$$

Step 9b: Rank 4 Capacity for Two-Stage Movements

The procedure for computing the total movement capacity for the subject movement considering the two-stage gap acceptance process is as follows. An adjustment factor a and an intermediate variable y are computed with Equation 20-55 and Equation 20-56, respectively.

Equation 20-55

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} \qquad \text{for } n_m > 0$$

Equation 20-56

$$y = \frac{c_{\mathrm{I}} - c_{m,x}}{c_{\mathrm{II}} - v_L - c_{m,x}}$$

where

n_m = number of storage spaces in the median;

c_{I} = movement capacity for the Stage I process (veh/h);

c_{II} = movement capacity for the Stage II process (veh/h);

v_L = major left-turn or U-turn flow rate, either $v_1 + v_{1U}$ or $v_4 + v_{4U}$ (veh/h); and

$c_{m,x}$ = capacity of subject movement, including the total conflicting flow rate for both stages of a two-stage gap acceptance process (from Step 9a).

The terms c_{I}, c_{II}, and $c_{m,x}$ are capacities after being adjusted for upstream signals and impedance. Use $v_1 + v_{1U}$ when considering Movements 7 and 8 and $v_4 + v_{4U}$ when considering Movements 10 and 11.

The total capacity c_T for the subject movement considering the two-stage gap acceptance process is computed by using Equation 20-57 and Equation 20-58 and incorporating the adjustment factors computed in Equation 20-55 and Equation 20-56.

For $y \neq 1$:

Equation 20-57

$$c_T = \frac{a}{y^{n_m+1} - 1}\left[y(y^{n_m} - 1)(c_{\mathrm{II}} - v_L) + (y - 1)c_{m,x}\right]$$

For $y = 1$:

Equation 20-58

$$c_T = \frac{a}{n_m + 1}\left[n_m(c_{\mathrm{II}} - v_L) + c_{m,x}\right]$$

Step 10: Final Capacity Adjustments

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

Where two or more movements share the same lane and cannot stop side by side at the stop line, Equation 20-59 is used to compute shared-lane capacity.

Equation 20-59

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}}$$

where

c_{SH} = capacity of the shared lane (veh/h),

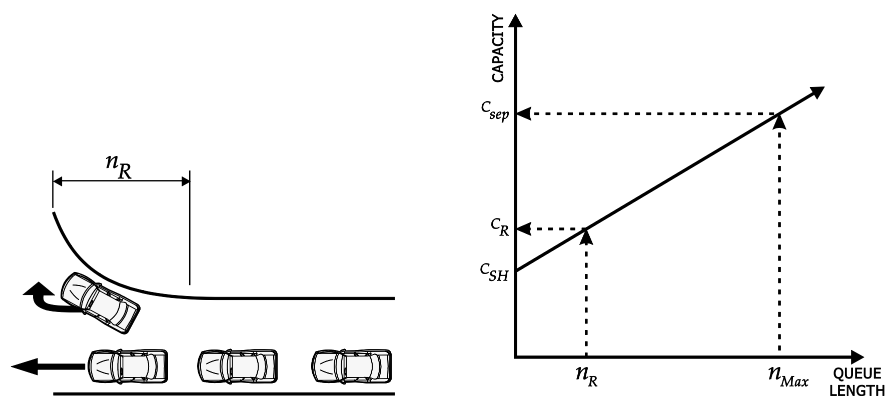
v_y = flow rate of the y movement in the subject shared lane (veh/h), and

$c_{m,y}$ = movement capacity of the y movement in the subject shared lane (veh/h).

Step 10b: Flared Minor-Street Lane Effects

To estimate the capacity of a flared right-turn lane (such as in Exhibit 20-17), the average queue length for each movement sharing the right lane on the minor-street approach must first be computed.

Exhibit 20-17
Capacity of a Flared-Lane Approach



This computation assumes the right-turn movement operates in one lane, and the other traffic in the right lane (upstream of the flare) operates in another, separate lane, as shown by Equation 20-60.

Equation 20-60

$$Q_{sep} = \frac{d_{sep} v_{sep}}{3{,}600}$$

where

Q_{sep} = average queue length for the movement considered as a separate lane (veh),

d_{sep} = control delay for the movement considered as a separate lane (as described in Step 11), and

v_{sep} = flow rate for the movement (veh/h).

Next, the required length of the storage area such that the approach would operate effectively as separate lanes is computed with Equation 20-61. This value is the maximum value of the queue lengths computed for each separate movement plus one vehicle.

$$n_{Max} = \max_i \left[\text{round}\left(Q_{sep,i} + 1\right)\right]$$ Equation 20-61

where

n_{Max} = length of the storage area such that the approach would operate as separate lanes;

$Q_{sep,i}$ = average queue length for movement *i* considered as a separate lane; and

round = round-off operator, rounding the quantity in parentheses to the nearest integer.

Next, the capacity of a separate lane condition c_{sep} must be computed and is assumed to be the capacity of right-turning traffic operating as a separate lane and the capacity of the other traffic in the right lane (upstream of the flare) operating as a separate lane. The capacity of a separate lane condition is calculated according to Equation 20-62.

$$c_{sep} = \min\left[c_R\left(1 + \frac{v_{L+TH}}{v_R}\right), c_{L+TH}\left(1 + \frac{v_R}{v_{L+TH}}\right)\right]$$ Equation 20-62

where

c_{sep} = sum of the capacity of the right-turning traffic operating as a separate lane and the capacity of the other traffic in the right lane (upstream of the flare) operating in a separate lane (veh/h),

c_R = capacity of the right-turn movement (veh/h),

c_{L+TH} = capacity of the through and left-turn movements as a shared lane (veh/h),

v_R = right-turn movement flow rate (veh/h), and

v_{L+TH} = through and left-turn movement combined flow rate (veh/h).

Finally, the capacity of the lane is computed, taking into account the flare. The capacity is interpolated as shown in Exhibit 20-17. A straight line is established by using the values of two points: (c_{sep}, n_{Max}) and (c_{SH}, 0). The interpolated value of the actual value of the flared-lane capacity c_R is computed with Equation 20-63.

$$c_R = \begin{cases} \left(c_{sep} - c_{SH}\right)\dfrac{n_R}{n_{Max}} + c_{SH} & \text{if } n_R \le n_{Max} \\ c_{sep} & \text{if } n_R > n_{Max} \end{cases}$$ Equation 20-63

where

c_R = actual capacity of the flared lane (veh/h),

c_{sep} = capacity of the lane if both storage areas were infinitely long (refer to Equation 20-62) (veh/h),

c_{SH} = capacity of the lane when all traffic shares one lane (veh/h), and

n_R = actual storage area for right-turning vehicles as defined in Exhibit 20-17.

The actual capacity c_R must be greater than c_{SH} but less than or equal to c_{sep}.

Step 11: Compute Movement Control Delay

The delay experienced by a motorist is related to factors such as control type, geometrics, traffic, and incidents. In the TWSC intersection methodology, only that portion of delay attributed to the STOP-control aspect of the intersection, referred to as control delay, is quantified.

Control delay includes delay due to deceleration to a stop at the back of the queue from free-flow speed, move-up time within the queue, stopped delay at the front of the queue, and delay due to acceleration back to free-flow speed. With respect to field measurements, control delay is defined as the total time that elapses from the time a vehicle stops at the end of the queue to the time the vehicle departs from the stop line. This total elapsed time includes the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position, including deceleration of the vehicle from free-flow speed to the speed of vehicles in the queue.

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

Average control delay for any particular minor movement is a function of the capacity of the approach and the degree of saturation. The analytical model used to estimate control delay (Equation 20-64) assumes demand is less than capacity for the period of analysis. If the degree of saturation is greater than about 0.9, average control delay is significantly affected by the length of the analysis period. In most cases, the recommended analysis period is 15 min. If demand exceeds capacity during a 15-min period, the delay results computed by the procedure may not be accurate. In this case, the period of analysis should be lengthened to include the period of oversaturation.

Equation 20-64

$$d = \frac{3{,}600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5$$

where

d = control delay (s/veh),

v_x = flow rate for movement x (veh/h),

$c_{m,x}$ = capacity of movement x (veh/h), and

T = analysis time period (0.25 h for a 15-min period) (h).

A constant value of 5 s/veh is used to reflect delay during deceleration to and acceleration from a stop.

The constant 5 s/veh is included in Equation 20-64 to account for the deceleration of vehicles from free-flow speed to the speed of vehicles in the queue and the acceleration of vehicles from the stop line to free-flow speed.

Step 11b: Compute Control Delay to Rank 1 Movements

The effect of a shared lane on the major-street approach where left-turning vehicles may block Rank 1 through or right-turning vehicles can be significant. If no exclusive left-turn pocket is provided on the major street, a delayed left-turning vehicle may block the Rank 1 vehicles behind it. This will delay not only Rank 1 vehicles but also lower-ranked movements. While the delayed Rank 1 vehicles are discharging from the queue formed behind a left-turning vehicle, they impede lower-ranked conflicting movements.

Field observations have shown that such a blockage effect is usually very small, because the major street usually provides enough space for the blocked Rank 1 vehicle to bypass the left-turning vehicle on the right. At a minimum, incorporating this effect requires estimating the proportion of Rank 1 vehicles being blocked and computing the average delay to the major-street left-turning vehicles that are blocking through vehicles.

In the simplest procedure, the proportion of Rank 1 major-street vehicles not being blocked (i.e., in a queue-free state) is given by $p^*_{0,j}$ in Equation 20-43 ($p^*_{0,j}$ should be substituted for the major left-turn factor $p_{0,j}$ in Equation 20-43 in computing the capacity of lower-ranked movements that conflict). Therefore, the proportion of Rank 1 vehicles being blocked is $1 - p^*_{0,j}$.

The average delay to Rank 1 vehicles is computed with Equation 20-65.

$$d_{Rank1} = \begin{cases} \dfrac{\left(1 - p^*_{0,j}\right) d_{M,LT} \left(\dfrac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} & N > 1 \\ \left(1 - p^*_{0,j}\right) d_{M,LT} & N = 1 \end{cases}$$

Equation 20-65

where

d_{Rank1} = delay to Rank 1 vehicles (s/veh),

N = number of through lanes per direction on the major street,

$p^*_{0,j}$ = proportion of Rank 1 vehicles not blocked (from Equation 20-43),

$d_{M,LT}$ = delay to major-street left-turning vehicles (from Equation 20-64) (s/veh),

$v_{i,1}$ = major-street through vehicles in shared lane (veh/h), and

$v_{i,2}$ = major-street turning vehicles in shared lane (veh/h).

On a multilane road, only the major-street volumes in the lane that may be blocked should be used in the computation as $v_{i,1}$ and $v_{i,2}$. On multilane roads, if it is assumed blocked Rank 1 vehicles do not bypass the blockage by moving into other through lanes (a reasonable assumption under conditions of high major-street flows), then $v_{i,1} = v_2/N$. Because of the unique characteristics associated with each site, the decision on whether to account for this effect is left to the analyst.

Step 12: Compute Approach and Intersection Control Delay

The control delay for all vehicles on a particular approach can be computed as the weighted average of the control delay estimates for each movement on the approach. Equation 20-66 is used for the computation.

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

Equation 20-66

where

d_A = control delay on the approach (s/veh);

d_r, d_t, d_l = computed control delay for the right-turn, through, and left-turn movements, respectively (s/veh); and

v_r, v_t, v_l = volume or flow rate of right-turn, through, and left-turn traffic on the approach, respectively (veh/h).

Similarly, the intersection control delay d_I can be computed with Equation 20-67.

Equation 20-67

$$d_I = \frac{d_{A,1}v_{A,1} + d_{A,2}v_{A,2} + d_{A,3}v_{A,3} + d_{A,4}v_{A,4}}{v_{A,1} + v_{A,2} + v_{A,3} + v_{A,4}}$$

where $d_{A,x}$ is the control delay on approach x (s/veh), and $v_{A,x}$ is the volume or flow rate on approach x (veh/h).

In applying Equation 20-66 and Equation 20-67, the delay for all Rank 1 major-street movements is assumed to be 0 s/veh. LOS is not defined for an overall intersection because major-street movements with 0 s of delay typically result in a weighted average delay that is extremely low. As such, total intersection control delay calculations are typically used only when comparing control delay among different types of traffic control, such as two-way STOP control versus all-way STOP control.

Step 13: Compute 95th Percentile Queue Lengths

Queue length is an important consideration at unsignalized intersections. Theoretical studies and empirical observations have demonstrated that the probability distribution of queue lengths for any minor movement at an unsignalized intersection is a function of the capacity of the movement and the volume of traffic being served during the analysis period. Equation 20-68 can be used to estimate the 95th percentile queue length for any minor movement at an unsignalized intersection during the peak 15-min period on the basis of these two parameters (*10*).

Equation 20-68

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\frac{c_{m,x}}{3{,}600}\right)$$

where

Q_{95} = 95th percentile queue (veh),

v_x = flow rate for movement x (veh/h),

$c_{m,x}$ = capacity of movement x (veh/h), and

T = analysis time period (0.25 h for a 15-min period) (h).

The mean queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest. The expected total delay (vehicle hours per hour) equals the expected number of vehicles in the average queue; that is, the total hourly delay and the average queue are numerically identical. For example, four vehicle hours per hour of delay can be used interchangeably with an average queue length of four vehicles during the hour.

4. EXTENSION TO THE MOTORIZED VEHICLE METHODOLOGY

INTRODUCTION

This section presents the details of incorporating pedestrian effects on motorized vehicle capacity into the motorized vehicle methodology. The steps below replace Steps 6 through 9 from Section 3.

REPLACEMENT STEPS TO INCORPORATE PEDESTRIAN EFFECTS ON MOTORIZED VEHICLE CAPACITY

Step 6: Compute Rank 1 Movement Capacities

Rank 1 major-street movements are assumed to be unimpeded by any movements of lower rank. Major-street movements of Rank 1 are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays occasionally occur, and they are accounted for by using adjustments provided later in this procedure.

For the purposes of this procedure, major-street movements of Rank 1 are assumed to be unimpeded by pedestrians at a TWSC intersection, even though research indicates some degree of Rank 1 vehicular yielding to pedestrians (see the pedestrian methodology in Section 5). The assumption that pedestrians do not impede Rank 1 major-street movements is a known limitation in the procedure.

Step 7: Compute Rank 2 Movement Capacities

Movements of Rank 2 (left turns from the major street and right turns from the minor street) must yield to conflicting major-street through and right-turning vehicular movements of Rank 1 as well as conflicting pedestrian movements of Rank 1. The movement capacity of each Rank 2 movement is equal to its potential capacity, factored by any impedance due to pedestrians.

Step 7a: Pedestrian Impedance

Minor vehicular movements must yield to conflicting pedestrian movements at a TWSC intersection. A factor accounting for pedestrian blockage is computed by Equation 20-69 on the basis of pedestrian volume, pedestrian walking speed, and width of the lane the minor movement is negotiating into.

$$f_{pb} = \frac{v_x \times \frac{w}{S_p}}{3{,}600}$$

Equation 20-69

where

f_{pb} = pedestrian blockage factor or proportion of time that one lane on an approach is blocked during 1 h;

v_x = number of groups of pedestrians, where x is Movement 13, 14, 15, or 16;

w = width of the lane the minor movement is negotiating into (ft); and

S_p = pedestrian walking speed, assumed to be 3.5 ft/s.

The pedestrian impedance factor for pedestrian movement x, $p_{p,x}$, is computed by Equation 20-70.

Equation 20-70

$$p_{p,x} = 1 - f_{pb}$$

Exhibit 20-18 shows that Rank 2 movements v_1 and v_4 must yield to pedestrian movements v_{16} and v_{15}, respectively. Exhibit 20-18 also shows that Rank 2 movement v_9 must yield to pedestrian movements v_{15} and v_{14}, and Rank 2 movement v_{12} must yield to pedestrian movements v_{16} and v_{13}. Rank 2 U-turn movements v_{1U} and v_{4U} are assumed to not yield to pedestrians crossing the major street, consistent with the assumptions stated previously for Rank 1 vehicles.

Exhibit 20-18
Relative Pedestrian–Vehicle Hierarchy for Rank 2 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_1	v_{16}	$p_{p,16}$
v_{1U}	—	—
v_4	v_{15}	$p_{p,15}$
v_{4U}	—	—
v_9	v_{15}, v_{14}	$(p_{p,15})(p_{p,14})$
v_{12}	v_{16}, v_{13}	$(p_{p,16})(p_{p,13})$

Step 7b: Movement Capacity for Major-Street Left-Turn Movements

Rank 2 major-street left-turn movements can be impeded by conflicting pedestrians. The movement capacity $c_{m,j}$ for major-street left-turn movements is computed with Equation 20-71.

Equation 20-71

$$c_{m,j} = c_{p,j} \times p_{p,i}$$

where j denotes movements of Rank 2 priority, i denotes movements of Rank 1 priority, and $c_{p,j}$ is the potential capacity of movement j.

Step 7c: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 minor-street right-turn Movements 9 and 12 is impeded by two conflicting pedestrian movements. The capacity adjustment factors are denoted by f_9 and f_{12} for minor-street right-turn Movements 9 and 12, respectively, and are given by Equation 20-72 and Equation 20-73, respectively.

Equation 20-72

$$f_9 = p_{p,15} \times p_{p,14}$$

Equation 20-73

$$f_{12} = p_{p,16} \times p_{p,13}$$

where

f_9, f_{12} = capacity adjustment factor for Rank 2 minor-street right-turn Movements 9 and 12, respectively; and

$p_{p,j}$ = probability that conflicting Rank 2 pedestrian movement j will operate in a queue-free state.

The movement capacity for minor-street right-turn movements is then computed with Equation 20-74.

Equation 20-74

$$c_{m,j} = c_{p,j} \times f_j$$

where

$c_{m,j}$ = movement capacity for Movements 9 and 12,

$c_{p,j}$ = potential capacity for Movements 9 and 12 (from Step 5), and

f_j = capacity adjustment factor for Movements 9 and 12.

Step 7d: Movement Capacity for Major-Street U-Turn Movements

This step is the same as Step 7c in Section 3.

Step 7e: Effect of Major-Street Shared Through and Left-Turn Lane

This step is the same as Step 7d in Section 3.

Step 8: Compute Rank 3 Movement Capacities

Rank 3 minor-street traffic movements (minor-street through movements at four-leg intersections and minor-street left turns at three-leg intersections) must yield to conflicting Rank 1 and Rank 2 movements. Not all gaps of acceptable length that pass through the intersection are normally available for use by Rank 3 movements because some of them are likely to be used by Rank 2 movements.

If the Rank 3 movement is a two-stage movement, the movement capacity for the one-stage movement is computed as an input to the two-stage calculation.

Step 8a: Pedestrian Impedance

Exhibit 20-19 shows that Rank 3 movements v_8 and v_{11} must yield to pedestrian movements v_{15} and v_{16}.

Exhibit 20-19 Relative Pedestrian–Vehicle Hierarchy for Rank 3 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_8	v_{15}, v_{16}	$(p_{p,15})(p_{p,16})$
v_{11}	v_{15}, v_{16}	$(p_{p,15})(p_{p,16})$

The pedestrian impedance factor for Rank 3 movements is computed according to Equation 20-69 and Equation 20-70.

Step 8b: Rank 3 Capacity for One-Stage Movements

This step is the same as Step 8a in Section 3, except that the capacity adjustment factor f_k for all movements k and for all Rank 3 movements is given by Equation 20-75.

$$f_k = \prod_j p_{0,j} \times p_{p,x}$$

Equation 20-75

where Π indicates the product of a series of terms, and

$p_{0,j}$ = probability that conflicting Rank 2 movement j will operate in a queue-free state,

$p_{p,x}$ = probability of pedestrian movements of Rank 1 or Rank 2 priority,

k = Rank 3 movements, and

x = 13, 14, 15, or 16 (pedestrian movements of both Rank 1 and Rank 2).

Step 8c: Rank 3 Capacity for Two-Stage Movements

This step is the same as Step 8b in Section 3.

Step 9: Compute Rank 4 Movement Capacities

Rank 4 movements (i.e., only the minor-street left turns at a four-leg intersection) can be impeded by all higher-ranked movements (Ranks 1, 2, and 3).

Step 9a: Pedestrian Impedance

Exhibit 20-20 shows that Rank 4 movement v_7 must yield to pedestrian movements v_{15} and v_{13}, and Rank 4 movement v_{10} must yield to pedestrian movements v_{16} and v_{14}.

Exhibit 20-20
Relative Pedestrian–Vehicle Hierarchy for Rank 4 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_7	v_{15}, v_{13}	$(p_{p,15})(p_{p,13})$
v_{10}	v_{16}, v_{14}	$(p_{p,16})(p_{p,14})$

The pedestrian impedance factor for Rank 4 movements is computed according to Equation 20-69 and Equation 20-70.

Step 9b: Rank 4 Capacity for One-Stage Movements

This step is the same as Step 9a in Section 3, except that the capacity adjustment factor for the Rank 4 minor-street left-turn movement can be computed by Equation 20-76.

Equation 20-76

$$f_l = p' \times p_{0,j} \times p_{p,x}$$

where

l = minor-street left-turn movement of Rank 4,

j = conflicting Rank 2 minor-street right-turn movement, and

$p_{p,x}$ = values shown in Equation 20-70 (the variable $p_{0,j}$ should be included only if movement j is identified as a conflicting movement).

Step 9c: Rank 4 Capacity for Two-Stage Movements

This step is the same as Step 9b in Section 3.

5. PEDESTRIAN MODE

SCOPE OF THE METHODOLOGY

This methodology applies to TWSC intersections and midblock crossings at which pedestrians cross up to four through lanes at a time on the major street. It is applied through a series of steps requiring input data related to vehicle and pedestrian volumes, geometric conditions, and motorist yield rates to pedestrians.

Spatial and Temporal Limits

This section's methodology applies to pedestrian crossings across an uncontrolled approach of a TWSC intersection or at a midblock location. The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This methodology produces the following performance measures:

- Average pedestrian delay, and
- Perception-based LOS based on the probability of crossing without delay and the type(s) of treatment(s) provided at the crossing.

Limitations of the Methodology

The pedestrian methodology's limitations differ from the limitations of the motorized vehicle mode because the methods were developed in separate research efforts. The pedestrian methodology does not apply to undivided streets with more than four through lanes, although it can accommodate up to four lanes in each direction separated by a median refuge. It does not account for interaction effects of upstream signalized intersections, it assumes random arrivals and equal lane distribution on the major street and, for one-stage crossings, it assumes equal directional distribution on the major street.

The methodology does not take into account pedestrian cross flows (i.e., pedestrian flows approximately perpendicular to and crossing another pedestrian stream), and it assumes the pedestrian will reach the crossing without delay from pedestrians traveling parallel to the major street. Under high pedestrian volumes, this assumption may not be reasonable.

The method is for steady state conditions (i.e., the demand and capacity conditions are constant during the analysis period); it is not designed to evaluate how fast or how often the facility transitions from one demand or capacity state to another.

Although the model only directly accounts for improved pedestrian satisfaction from selected pedestrian safety countermeasures, the effects of other countermeasures on satisfaction can be indirectly incorporated.

The pedestrian crossing LOS model indicates improved pedestrian satisfaction when specific pedestrian safety countermeasures exist: marked crosswalks, median refuge islands, rectangular rapid-flashing beacons (RFFBs), or a combination of these. The research that developed this model (*11*) did not study the full range of safety countermeasures in use and therefore may underpredict pedestrian satisfaction with other types of safety countermeasures. However, the effects of other types of safety countermeasures are indirectly accounted for through improvements in driver yielding, reductions in crossing length, or both.

Alternative Tool Considerations

This section offers a method for estimating the delay and LOS for pedestrians crossing a major street at a TWSC intersection or midblock location. Some simulation-based tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with pedestrians, but for most purposes the HCM procedure produces an acceptable approximation.

The identified limitations for this chapter are shown in Exhibit 20-21, along with the potential for improved treatment by alternative tools.

Exhibit 20-21
Limitations of the HCM TWSC Pedestrian Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Crossing more than four through lanes in one stage	Simulation tools may be able to accommodate larger lane configurations.
Effects of upstream intersections	Simulation tools can include an unsignalized intersection explicitly within a signalized arterial or network.
Pedestrian cross flows parallel to the major street that impede pedestrian crossings across the major street	Simulation tools that model pedestrian flows explicitly may be able to capture this effect.
Non–steady state conditions for demand and capacity	Most alternative tools provide for multiperiod variation of demand and, in some cases, capacity.

REQUIRED INPUT DATA AND SOURCES

Exhibit 20-22 lists the information necessary to apply the pedestrian methodology and suggests potential sources for obtaining these data. It also suggests default values for use when specific information is not available.

Exhibit 20-22
Required Input Data, Potential Data Sources, and Default Values for TWSC Pedestrian Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Number of lanes on the major street	Design plans, road inventory	Must be provided
Crosswalk length (ft)	Design plans, road inventory	Must be provided
Crosswalk width (ft)	Design plans, road inventory	Must be provided
Presence of crossing treatments: marked crosswalk, median refuge, rectangular rapid-flashing beacon	Design plans, road inventory	Must be provided
	Demand Data	
Pedestrian flow rate (p/s)	Field data, modeling	Must be provided
Presence of pedestrian platooning	Field data, modeling	Must be provided
Conflicting vehicular flow rate (veh/s)	Field data, modeling	Must be provided
Pedestrian walking speed (ft/s)	Field data	See discussion below
Pedestrian start-up time and end clearance time (s)	Field data	See discussion below
Mean motorist yielding rate to pedestrians	Field data, literature	Must be provided[a]

Note: [a] Sample values from the literature are provided in this section (see Exhibit 20-24).

The choice of pedestrian walking speed depends on the analysis purpose:

- For estimating average pedestrian delay (e.g., as part of a person delay analysis) or pedestrian LOS under existing conditions, a locally measured average walking speed for uncontrolled crossings is recommended. In the absence of local data, research (*11–13*) has found an average pedestrian speed while crossing of 4.7 ft/s, which is slightly higher than the average pedestrian speed on sidewalks of 4.4 ft/s given in Chapter 18.

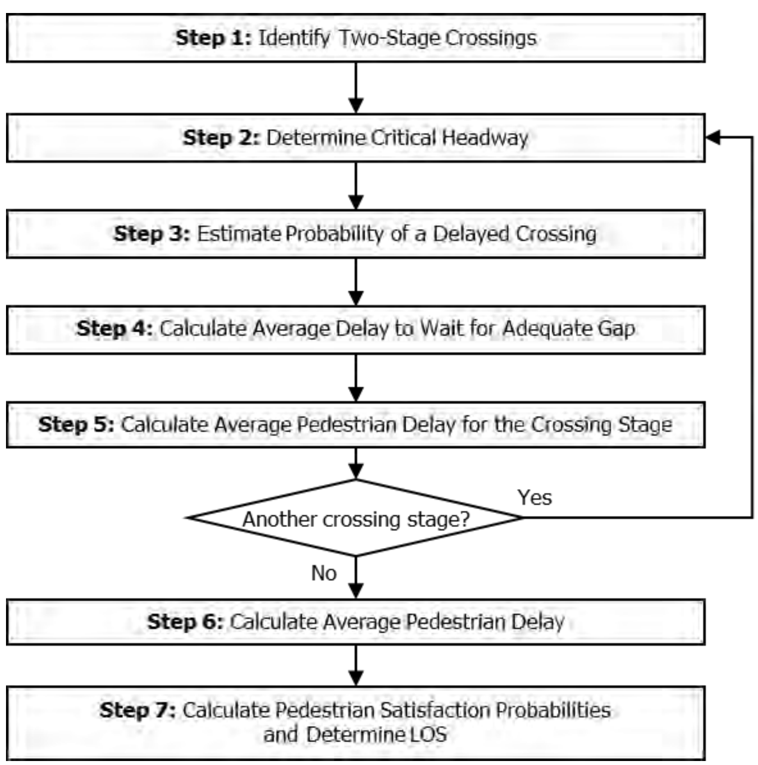
- For planning and design purposes, for example to assess the adequacy of the crossing to accommodate pedestrians with a variety of abilities, a walking speed of 3.5 ft/s, representative of a 15th-percentile pedestrian, may be appropriate.

The research that developed this method (*11*) found that field-measured values of average delay best matched the estimated delay when the pedestrian start-up and end clearance time was 0 s. This value implies that pedestrians anticipate the arrival of an adequate gap (i.e., they do not require any start-up time) and start immediately upon its arrival. It also implies that pedestrians do not require any end clearance time. However, it is more likely that the vehicles defining the start and end of the adequate gap are often not traveling in the first and last lanes, respectively, crossed by the pedestrian (hence, crossing safety is assured spatially by lane separation rather than temporally by a second or two of clearance time). As always, the use of local values is encouraged when available. For design purposes, a start-up and end clearance time value of 3.0 s provides a more conservative estimate.

COMPUTATIONAL STEPS

The required steps are illustrated in Exhibit 20-23.

Exhibit 20-23
TWSC Pedestrian Methodology



Step 1: Identify Two-Stage Crossings

When a median refuge is available, pedestrians typically cross in two stages, similar to the two-stage movement described for motorized vehicles earlier in this chapter. Determination of whether a median refuge exists may require engineering judgment; however, two-way left-turn lanes and raised medians narrower than 6 ft are not treated as median refuges for the purposes of this procedure. The main issue to determine is whether pedestrians cross the traffic streams in one or two stages. When pedestrians cross in two stages, pedestrian delay should be estimated separately for each stage of the crossing by using the procedures described in Steps 2 through 6, separating the conflicting vehicular volume accordingly. The pedestrian delay for each stage should be summed to establish the average pedestrian delay associated with the entire crossing.

Step 2: Determine Critical Headway

Critical headway for pedestrians is similar to critical headway for motorized vehicles.

The procedure for estimating critical headway for pedestrians is similar to that described for motorized vehicles. The critical headway is the minimum time interval in seconds below which a pedestrian will not attempt to begin crossing the street. Pedestrians use their judgment to determine whether the available headway between conflicting vehicles is long enough for a safe crossing. If the available headway is greater than the critical headway, it is assumed the pedestrian will cross, but if the available headway is less than the critical headway, it is assumed the pedestrian will not cross.

For a single pedestrian, critical headway is computed with Equation 20-77.

Equation 20-77

$$t_c = \frac{L}{S_p} + t_s$$

where

t_c = critical headway for a single pedestrian (s),

S_p = average pedestrian walking speed (ft/s),

L = crosswalk length (ft), and

t_s = pedestrian start-up time and end clearance time (s).

Groups of pedestrians require computation of their spatial distribution.

If groups of pedestrians are observed crossing in the field (i.e., a platoon, or more than one pedestrian crossing at a time), then the spatial distribution of pedestrians should be computed with Equation 20-78. The spatial distribution of pedestrians represents the number of rows of pedestrians waiting to cross, with the first row in position to cross and subsequent rows lined behind the first row. If the crosswalk is wide enough to accommodate a group of pedestrians traveling side-by-side without needing to also travel behind one another, then the spatial distribution of pedestrians equals one row. If no pedestrian grouping is observed, the spatial distribution of pedestrians is assumed to be one row.

Equation 20-78

$$N_p = \max\left[\frac{8.0\ N_c}{W_c}, 1.0\right]$$

where

N_p = spatial distribution of pedestrians (pedestrian rows),

N_c = total number of pedestrians in the crossing platoon (from Equation 20-79) (p),

W_c = crosswalk width (ft), and

8.0 = default clear effective width used by a single pedestrian to avoid interference when passing other pedestrians (ft).

To compute spatial distribution, the analyst must make field observations or estimate the platoon size by using Equation 20-79.

Equation 20-79

$$N_c = \frac{v_p e^{v_p t_c} + v e^{-v t_c}}{(v_p + v) e^{(v_p - v) t_c}}$$

where

N_c = total number of pedestrians in the crossing platoon (p),

v_p = pedestrian flow rate (p/s),

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings), and

t_c = single pedestrian critical headway (s).

The value of v should always be positive to avoid division-by-zero errors in subsequent equations; a minimum value of 0.0001 veh/s is recommended in Equation 20-79 and subsequent equations in this methodology that use v.

The group critical headway is the critical headway needed to accommodate a group of pedestrians. The group critical headway is determined with Equation 20-80.

Equation 20-80

$$t_{c,G} = t_c + 2(N_p - 1)$$

where

$t_{c,G}$ = group critical headway (s),

t_c = critical headway for a single pedestrian (s), and

N_p = spatial distribution of pedestrians (pedestrian rows).

Step 3: Estimate Probability of a Delayed Crossing

On the basis of the calculation of the critical headway $t_{c,G}$, the probability that a pedestrian will not incur any crossing delay is equal to the likelihood that a pedestrian will encounter a gap greater than or equal to the critical headway immediately upon arrival at the intersection.

Assuming random arrivals of vehicles on the major street and equal distribution of vehicles among all through lanes on the major street, the likelihood that a gap in a given lane does not exceed the critical headway is as shown in Equation 20-81. Because traffic is assumed to be distributed independently in each through lane, Equation 20-82 shows the probability that a pedestrian incurs nonzero delay at a TWSC crossing.

Equation 20-81

$$P_b = 1 - e^{\frac{-t_{c,G}v}{N_L}}$$

Equation 20-82

$$P_d = 1 - (1 - P_b)^{N_L}$$

where

P_b = probability of a blocked lane,

P_d = probability of a delayed crossing,

N_L = number of through lanes crossed,

$t_{c,G}$ = group critical headway (s), and

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings).

Step 4: Calculate Average Delay to Wait for Adequate Gap

Research indicates average delay to pedestrians at unsignalized crossings, assuming no motor vehicles yield and the pedestrian is forced to wait for an adequate gap, depends on the critical headway, the vehicular flow rate of the subject crossing, and the mean vehicle headway (*14*). The average delay per pedestrian to wait for an adequate gap is given by Equation 20-83.

Equation 20-83

$$d_g = \frac{1}{v}\left(e^{vt_{c,G}} - vt_{c,G} - 1\right)$$

where

d_g = average pedestrian gap delay (s),

$t_{c,G}$ = group critical headway (s), and

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings).

The average delay for any pedestrian who is unable to cross immediately upon reaching the intersection (e.g., any pedestrian experiencing nonzero delay) is thus a function of P_d and d_g, as shown in Equation 20-84.

Equation 20-84

$$d_{gd} = \frac{d_g}{P_d}$$

where

d_{gd} = average gap delay for pedestrians who incur nonzero delay,

d_g = average pedestrian gap delay (s), and

P_d = probability of a delayed crossing.

Step 5: Calculate Average Pedestrian Delay for the Crossing Stage

When a pedestrian arrives at a crossing and finds an inadequate gap, that pedestrian is delayed until one of two situations occurs: (*a*) a gap greater than the critical headway is available, or (*b*) motor vehicles yield and allow the pedestrian to cross. Equation 20-83 estimates pedestrian delay when motorists on the major approaches do not yield to pedestrians. When motorist yield rates are significantly higher than zero, pedestrians will experience considerably less delay than that estimated by Equation 20-83.

In the United States, motorists are legally required to yield to pedestrians, under most circumstances, in both marked and unmarked crosswalks. However, actual motorist yielding behavior varies considerably. Exhibit 20-24 provides information on average motorist responses to typical pedestrian crossing treatments summarized from a number of research efforts (*11*, *12*, *15–30*). As indicated by the large range of observed yielding rates for many pedestrian crossing safety countermeasures, motorist yield rates are influenced by a range of factors. These factors include roadway geometry, travel speeds, isolated vs. corridor- or citywide pedestrian crossing treatments, local culture, and law enforcement practices. In nearly all cases, safety countermeasures improved yielding rates at a given site compared to the "before" condition (e.g., crosswalk markings only). As always, practitioners should supplement or replace these values with local knowledge and engineering judgment. Furthermore, decisions to install a particular treatment should also consider the treatment's effect on safety and whether site-specific conditions make the treatment inappropriate for that location.

Depending on the crossing treatment and other factors, motorist behavior varies significantly.

Exhibit 20-24
Effect of Pedestrian Crossing Treatments on Motorist Yield Rates

Crossing Treatment	Yield Rate (%) Average	Yield Rate (%) Range	Sample Size (sites)
No treatment (unmarked)	24	0–100	37
Crosswalk markings only (any type)	33	0–95	58
Crosswalk markings, plus:			
Pedestal-mounted flashing beacon	26	0–52	2
Overhead sign	35	12–57	2
Overhead flashing beacon (push-button activation)	51	13–91	14
Overhead flashing beacon (passive activation)	73	61–76	29
In-roadway warning lights	58	53–65	11
Median refuge island	60	0–100	21
Pedestrian crossing flags	74	72–80	6
In-street pedestrian crossing signs	76	35–88	20
Rectangular rapid-flashing beacon (RFFB)	82	31–100	64
School crossing guard	86	—	1
School crossing guard and RFFB	92	—	1
Pedestrian hybrid beacon (HAWK)	91	73–99	37
Mid-block crossing signals, half signals	98	94–100	13

Sources: Ryus et al. (*11*), Fitzpatrick et al. (*12*), Huang et al. (*15*), Turner et al. (*16*), Banerjee and Ragland (*17*), Ellis Jr. et al. (*18*), Shurbutt et al. (*19*), Mitman et al. (*20*), Pécheaux et al. (*21*), Mitman et al. (*22*), Ross et al. (*23*), Brewer and Fitzpatrick (*24*), Fitzpatrick et al. (*25*), Nemeth et al. (*26*), Yang et al. (*27*), Zheng and Elefteriatou (*28*), Schneider et al. (*29*), Al-Kaisy et al. (*30*), and Hockmuth and Van Houten (*31*).

It is possible for pedestrians to incur less actual delay than d_g because of yielding vehicles. The likelihood of this situation occurring is a function of vehicle volumes, motorist yield rates, and number of upstream lanes on the major street. Consider a pedestrian waiting for a crossing opportunity at a TWSC intersection or midblock crossing, with vehicles in each conflicting through lane arriving every h seconds. On average, a potential yielding event will occur every h seconds. As vehicles are assumed to arrive randomly, each potential yielding event is considered to be independent. Turn lanes are usually fed from upstream through lanes (i.e., a vehicle moves from an upstream through lane into a turn pocket) and therefore are not counted separately from the upstream through lanes except when a through lane is dropped as a turn lane at an intersection.

For each potential yielding event, each through lane is in one of two states:

1. Clear—no vehicles are arriving within the critical headway window, or
2. Blocked—a vehicle is arriving within the critical headway window.

If all through lanes are clear when a pedestrian arrives, the pedestrian experiences no delay and can cross immediately. If at least one lane is blocked, the pedestrian may only cross immediately if vehicles in each blocked lane choose to yield. If one or more blocking vehicles do not yield, the pedestrian must wait an additional h seconds for the next potential yielding event. On average, this process will be repeated until the wait exceeds the expected delay required for an adequate gap in traffic (d_{gd}), at which point the average pedestrian will receive an adequate gap in traffic and will be able to cross the street without having to depend on yielding motorists.

Average pedestrian delay can be calculated with Equation 20-85, where the first term in the equation represents expected delay from crossings occurring when motorists yield, and the second term represents expected delay from crossings when pedestrians wait for an adequate gap.

Equation 20-85

$$d_{p,s} = \sum_{i=0}^{n} h(i - 0.5)P(Y_i) + \left(P_d - \sum_{i=0}^{n} P(Y_i)\right) d_{gd}$$

where

$d_{p,s}$ = average pedestrian delay for crossing stage s (s);

i = potential yielding event (i = 0 to n);

h = average headway of those headways less than group critical headway (s), from Equation 20-86;

$P(Y_i)$ = probability that motorists yield to pedestrian on potential yielding event i;

P_d = probability of a potentially delayed crossing; and

n = average number of potential yielding events before an adequate gap is available = int(d_{gd}/h).

Note the possibility for n to have large values with high traffic volumes. For example, n = 148 when v × $t_{c,G}$ = 5.

Equation 20-86 computes the average headway of those headways less than group critical headway h (*32*).

Equation 20-86

$$h = \frac{1/v - (t_{c,G} + 1/v)e^{-v\, t_{c,G}}}{1 - e^{-v\, t_{c,G}}}$$

where

h = average headway of those headways less than group critical headway (s),

$t_{c,G}$ = group critical headway (s), and

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings).

Equation 20-85 requires the calculation of $P(Y_i)$. The probabilities $P(Y_i)$ that motorists will yield for a given potential yielding event are considered below for pedestrian crossings of one, two, three, and four through lanes. The probability of yielding $P(Y_0)$ when there are no potential yielding events (i.e., $n = 0$) equals 0.0 regardless of how many lanes are crossed.

One-Lane Crossing

Under the scenario in which a pedestrian crosses one through lane, $P(Y_i)$ is found simply. When $i = 1$, $P(Y_i)$ is equal to the probability of a delayed crossing P_d multiplied by the motorist yield rate M_y, as given by Equation 20-87:

$$P(Y_1) = P_d M_y$$

Equation 20-87

For $i = 2$, $P(Y_i)$ is equal to M_y multiplied by the probability that the second potential yielding event occurs (i.e., that the pedestrian did not cross on the first potential yielding event), $P_d^*(1 - M_y)$. Equation 20-88 gives $P(Y_i)$ for any i.

$$P(Y_i) = P_d M_y \left(1 - M_y\right)^{i-1}$$

Equation 20-88

where

M_y = motorist yield rate (decimal) ($M_y \leq 0.9999$), and

i = potential yielding event ($i = 0$ to n).

Because the value 0^0 is undefined in calculation tools, a 100% motorist yielding rate should be reduced to 99.99% (i.e., 0.9999) for use in Equation 20-88.

Two-Lane Crossing

For a two-lane pedestrian crossing, $P(Y_i)$ requires either (a) motorists in both lanes to yield simultaneously if both lanes are blocked or (b) a single motorist to yield if only one lane is blocked. Because these cases are mutually exclusive, where $i = 1$, $P(Y_i)$ is given by Equation 20-89.

$$P(Y_1) = 2P_b(1 - P_b)M_y + P_b^2 M_y^2$$

Equation 20-89

where P_b is the probability of a blocked lane.

Equation 20-90 shows $P(Y_i)$ where i is greater than one. Equation 20-90 is equivalent to Equation 20-89 if $P(Y_0)$ is set to equal zero.

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{\left(2P_b[1 - P_b]M_y\right) + \left(P_b^2 M_y^2\right)}{P_d}\right]$$

Equation 20-90

Three-Lane Crossing

A three-lane crossing follows the same principles as a two-lane crossing. The probability of all blocking vehicles yielding on the first potential yielding event is given by Equation 20-91.

$$P(Y_1) = P_b^3 M_y^3 + 3P_b^2(1 - P_b)M_y^2 + 3P_b(1 - P_b)^2 M_y$$

Equation 20-91

Equation 20-92 shows the calculation for $P(Y_i)$ where i is greater than one.

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{P_b^3 M_y^3 + 3P_b^2(1 - P_b)M_y^2 + 3P_b(1 - P_b)^2 M_y}{P_d}\right]$$

Equation 20-92

Four-Lane Crossing

A four-lane crossing follows the same principles as above. The probability of all blocking vehicles yielding on the first potential yielding event is given by Equation 20-93.

Equation 20-93

$$P(Y_1) = P_b^4 M_y^4 + 4P_b^3(1 - P_b)M_y^3 + 6P_b^2(1 - P_b)^2 M_y^2 + 4P_b(1 - P_b)^3 M_y$$

Equation 20-94 shows the calculation for $P(Y_i)$ where i is greater than one.

Equation 20-94

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right] \times \left[\frac{P_b^4 M_y^4 + 4P_b^3(1 - P_b)M_y^3 + 6P_b^2(1 - P_b)^2 M_y^2 + 4P_b(1 - P_b)^3 M_y}{P_d}\right]$$

Step 6: Calculate Average Pedestrian Delay

In the case of a one-stage crossing, the average pedestrian delay d_p is the same as the average pedestrian delay for the first crossing stage $d_{p,1}$. Equation 20-95 gives the average pedestrian delay for multiple-stage crossings.

Equation 20-95

$$d_p = \sum_{s=1}^{n_{cs}} d_{p,s}$$

where

d_p = average pedestrian control delay (s),

$d_{p,s}$ = average pedestrian delay for crossing stage s (s), and

n_{cs} = number of crossing stages.

Exhibit 20-25 provides interpretations of different ranges of pedestrian delay.

Exhibit 20-25 Interpretation of Different Levels of Pedestrian Control Delay

Control Delay (s/p)	Comments
0–5	Usually no conflicting traffic
5–10	Occasionally some delay due to conflicting traffic
10–20	Delay noticeable to pedestrians, but not inconveniencing
20–30	Delay noticeable and irritating, increased likelihood of risk taking
30–45	Delay approaches tolerance level, risk-taking behavior likely
>45	Delay exceeds tolerance level, high likelihood of pedestrian risk taking

Step 7: Calculate Pedestrian Satisfaction Probabilities and Determine LOS

The service measure for a pedestrian crossing at a mid-block or TWSC intersection location is based on the predicted average proportion of pedestrians who would say they were “dissatisfied” or worse with their crossing experience. The research that developed this portion of the methodology (*11*) surveyed actual pedestrians using four potential levels of satisfaction: very satisfied, satisfied, dissatisfied, and very dissatisfied. These levels have been condensed to two levels—satisfied or dissatisfied—for ease of implementation.

Equation 20-96 estimates the odds that pedestrians would be satisfied with their crossing experience relative to being dissatisfied.

Equation 20-96

$$O(S/D) = \exp(0.9951 - 0.0438V_{KAADT} + 1.9572I_{RRFB} + 0.9843I_{MC} + 1.5496I_{MR} - 1.9059I_{NY})$$

where

$O(S/D)$ = odds that a pedestrian would be satisfied with their crossing experience relative to being dissatisfied;

exp = exponential function;

V_{KAADT} = annual average daily traffic of the street being crossed (1000s of veh);

I_{RRFB} = indicator variable for the presence of a rectangular rapid-flashing beacon (RRFB) at the crossing (1 = present, 0 = not present);

I_{MC} = indicator variable for the presence of a marked crosswalk (1 = present, 0 = not present);

I_{MR} = indicator variable for the presence of a median refuge (1 = present, 0 = not present); and

I_{NY} = indicator variable for the pedestrian experiencing a vehicle not yielding while using the crossing (1 = not yielding, 0 = yielding).

Equation 20-97 estimates the probability of a given pedestrian being satisfied with their crossing. The probability of a given pedestrian being dissatisfied is then one minus the probability of being satisfied, as shown by Equation 20-98.

$$P(S) = \frac{O(S/D)}{O(S/D) + 1}$$

Equation 20-97

$$P(D) = 1 - P(S)$$

Equation 20-98

where

$P(S)$ = probability that a pedestrian would be satisfied with their crossing experience (decimal),

$P(D)$ = probability that a pedestrian would be dissatisfied with their crossing experience (decimal), and

all other terms are as defined previously.

When $I_{NY} = 0$, Equation 20-97 and Equation 20-98 produce the probabilities of being satisfied and dissatisfied when the pedestrian is not delayed while using the crossing (i.e., either a sufficient gap exists when the pedestrian arrives to allow an immediate crossing, or all blocking vehicles yield to the pedestrian). Similarly, when $I_{NY} = 1$, these equations produce the probabilities of being satisfied and dissatisfied when the pedestrian is delayed while using the crossing.

The probability of a non-delayed crossing is the sum of the probability of a sufficient gap existing to allow an immediate crossing when the pedestrian arrives (i.e., one minus the probability of a delayed crossing), plus the proportion of the potentially delayed crossings in which all blocking vehicles yield to the pedestrian on the first potential yielding event. Equation 20-99 calculates the probability of a non-delayed crossing.

$$P_{nd} = (1 - P_d) + P_d P(Y_1)$$

Equation 20-99

where

P_{nd} = probability of a non-delayed crossing (decimal);

P_d = probability of a potentially delayed crossing (decimal), from Equation 20-82; and

$P(Y_1)$ = probability of all blocking vehicles yielding on the first potential yielding event (decimal), from Equation 20-87, Equation 20-89, Equation 20-91, or Equation 20-93 for one-, two-, three, or- four-lane crossings, respectively.

Over the course of the analysis period, a proportion of crossing pedestrians P_{nd} will experience no delay while using the crossing; the number of "satisfied" and "dissatisfied" ratings from these pedestrians will be in proportion to the respective satisfaction probabilities when no delay occurs. Similarly, the remaining proportion of crossing pedestrians P_d will be delayed while using the crossing; the number of ratings in each category from these pedestrians will be in proportion to the respective satisfaction probabilities when a delay occurs. The overall proportion of "dissatisfied" ratings is therefore the volume-weighted average of the probabilities of being "dissatisfied" under no-delay and delay conditions, as given by Equation 20-100.

Equation 20-100

$$P_D = P_{nd} P(D, \text{no delay}) + (1 - P_{nd}) P(D, \text{delay})$$

where

P_D = average proportion of "dissatisfied" ratings for the crossing (decimal),

P_{nd} = probability of a non-delayed crossing (decimal),

$P(D, \text{no delay})$ = probability of a "dissatisfied" rating when no delay occurs (decimal), and

$P(D, \text{delay})$ = probability of a "dissatisfied" rating when a delayed crossing occurs (decimal).

The value of P_D can be used with Exhibit 20-3 to determine the crossing's LOS.

6. BICYCLE MODE

As of the publication of this edition of the HCM, no methodology specific to bicyclists has been developed to assess the performance of bicyclists at TWSC intersections, as few data are available in the United States to support model calibration or LOS definitions. Depending on individual comfort level, ability, geometric conditions, and traffic conditions, a bicyclist may travel through the intersection either as a motor vehicle or as a pedestrian. Critical headway distributions have been identified in the research (*33*, *34*) for bicycles crossing two-lane major streets. Data on critical headways for bicycles under many circumstances are not readily available, however. Bicycles also differ from motor vehicles in that they normally do not queue linearly at a STOP sign. Instead, multiple bicycles often use the same gap in the vehicular traffic stream. This practice probably affects the determination of bicycle follow-up time. This phenomenon and others described in this section have not been adequately researched and are not explicitly included in the methodology.

7. APPLICATIONS

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary engineering analysis.

Operational Analysis

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement in vehicles per hour, percentage of heavy vehicles for each approach, peak hour factor for all demand volumes, lane configurations, specific geometric conditions, and upstream signal information. The outputs of an operational analysis are estimates of capacity, control delay, and queue lengths. The steps of the methodology, described in this chapter's methodology section, are followed directly without modification.

Design Analysis

The operational analysis described earlier in this chapter can be used for design purposes by using a given set of traffic flow data and iteratively determining the number and configuration of lanes that would be required to produce a given LOS.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of a TWSC intersection. To estimate LOS for a future time horizon, a planning analysis based on the operational method is used. The planning method uses all the geometric and traffic flow data required for an operational analysis, and the computations are identical. However, input variables for percentage of heavy vehicles and peak hour factor are typically estimated (or defaults are used) when planning applications are performed.

EXAMPLE PROBLEMS

Section 2 of Chapter 32, STOP-Controlled Intersections: Supplemental, provides five example problems that illustrate each of the computational steps involved in applying the motorized vehicle method:

1. Analyze a TWSC intersection with three legs,
2. Analyze a pedestrian crossing at a TWSC intersection,
3. Analyze a TWSC intersection with flared approaches and median storage,
4. Analyze a TWSC intersection within a signalized urban street segment, and
5. Analyze a TWSC intersection on a six-lane street with U-turns and pedestrians.

EXAMPLE RESULTS

Analysis of TWSC intersections is commonly performed to determine whether an existing intersection or driveway can remain as a TWSC intersection or whether additional treatments are necessary. These treatments, including geometric modifications and changes in traffic control, are discussed in other references, including the presentation of traffic signal warrants in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (*35*). This section discusses two common situations analysts face: the analysis of shared versus separate lanes and the interpretation of LOS F.

Analysis of Shared Versus Separate Lanes

Interpretation of the effects of shared lanes should consider both delay associated with individual movements and delay associated with all vehicles on a given approach.

Some movements, most often left-turn movements, can sometimes have a poorer LOS when given a separate lane than when they share a lane with another movement (usually a through movement). This is not inconsistent in terms of the stated criteria. Left-turn movements will generally experience longer control delays than other movements because of the nature and priority of the movement. The control delay for left turns in a shared lane may be less than the control delay for left turns in a separate lane. However, if delay for all vehicles on the approach or at the intersection is considered, providing separate lanes will result in lower total delay.

Interpretation of LOS F

LOS F occurs when there are not enough gaps of suitable size to allow minor-street vehicles to enter or cross through traffic on the major street; this results in long average control delays (greater than 50 s/veh). Depending on the demand on the approach, long queues on the minor approaches may result. The method, however, is based on a constant critical headway.

LOS F may also appear in the form of drivers on the minor street selecting smaller-than-usual gaps. In such cases, safety issues may occur, and some disruption to the major traffic stream may result. With lower demands, LOS F may not always result in long queues.

At TWSC intersections, the critical movement, often the minor-street left turn, may control the overall performance of the intersection. The lower threshold for LOS F is set at 50 s of delay per vehicle. In some cases, the delay equations will predict delays greater than 50 s for minor-street movements under very low-volume conditions on the minor street (fewer than 25 veh/h). On the basis of the first term of the delay equation, the LOS F threshold is reached with a movement capacity of approximately 85 veh/h or less, regardless of the minor-street movement volume.

This analysis procedure assumes random arrivals on the major street. For a typical major street with two lanes in each direction and an average traffic volume in the range of 15,000 to 20,000 veh/day (roughly equivalent to a peak hour flow rate of 1,500 to 2,000 veh/h), the delay equation will predict greater than 50 s of delay (LOS F) for many urban TWSC intersections that allow minor-street left-turn movements. LOS F will be predicted regardless of the volume of minor-street left-turning traffic. Even with a LOS F estimate, most low-volume minor-street approaches would not meet any of the volume or delay warrants for signalization noted in the *Manual on Uniform Traffic Control Devices* (*35*). As a result, analysts who use the HCM LOS thresholds as the sole measure to determine the design adequacy of TWSC intersections should do so with caution.

In evaluating the overall performance of TWSC intersections, it is important to consider measures of effectiveness such as volume-to-capacity ratios for individual movements, average queue lengths, and 95th percentile queue lengths in addition to considering delay. By focusing on a single measure of effectiveness for the worst movement only, such as delay for the minor-street left turn, users may make less effective traffic control decisions.

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Enhancing Pedestrian Volume Estimation and Developing HCM Pedestrian Methodologies for Safe and Sustainable Communities

Chapter 30: Urban Street Segments: Supplemental

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This chapter contains revisions to Example Problem 2: Pedestrian LOS (Section 8) related to the changes to the roadway crossing difficulty factor in Chapter 18. These revisions were developed through NCHRP Project 17-87.

CHAPTER 30
URBAN STREET SEGMENTS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 30 is the supplemental chapter for Chapter 18, Urban Street Segments, which is found in Volume 3 of the *Highway Capacity Manual* (HCM). This chapter presents detailed information about the following aspects of the Chapter 18 motorized vehicle methodology:

- The adjustments made to the input vehicular demand flow rates at signalized boundary intersections so that they reasonably reflect actual operating conditions during the analysis period,
- The process for analyzing vehicular traffic flow on a segment bounded by signalized intersections, and
- The process for estimating through-vehicle delay due to vehicle turning movements at unsignalized midsegment access points.

This chapter provides a simplified version of the Chapter 18 motorized vehicle methodology that is suitable for planning applications. It describes techniques for measuring free-flow speed and average travel speed in the field and provides details about the computational engine that implements the Chapter 18 motorized vehicle methodology. Chapter 30 provides four example problems that demonstrate the application of the motorized vehicle, pedestrian, bicycle, and transit methodologies to an urban street segment. Finally, the chapter provides an overview of the methodology for evaluating the performance of the motor vehicle mode on an urban street segment bounded by one or more roundabouts.

2. TRAFFIC DEMAND ADJUSTMENTS

This section describes adjustments made to the input vehicular demand flow rates at signalized boundary intersections so that they reasonably reflect actual operating conditions during the analysis period. These adjustments have no effect if existing vehicular flow rates are accurately quantified for the subject segment and all movements operate below their capacity. However, if the demand flow rate for any movement exceeds its capacity or if there is disagreement between the count of vehicles entering and the count exiting the segment, some movement flow rates will need to be adjusted for accurate evaluation of segment operation.

This section describes two procedures that check the input flow rates and make adjustments if necessary. These procedures are

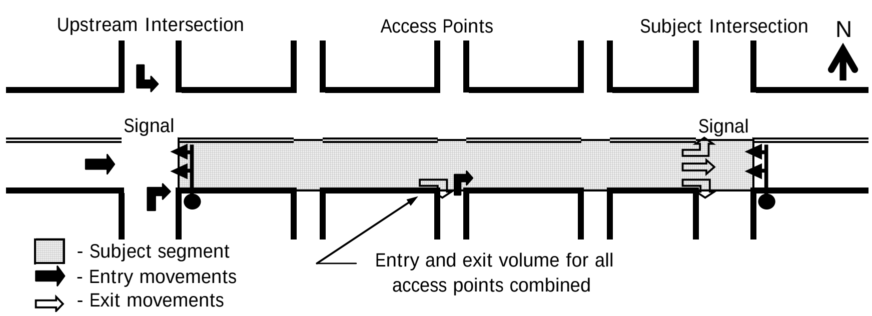
- Capacity constraint and volume balance and
- Origin–destination distribution.

These procedures can be extended to the analysis of unsignalized boundary intersections; however, the mechanics of this extension are not described.

CAPACITY CONSTRAINT AND VOLUME BALANCE

This subsection describes the procedure for determining the turn movement flow rates at each intersection along the subject urban street segment. The analysis is separately applied to each travel direction and proceeds in the direction of travel. The procedure consists of a series of steps that are completed in sequence for the entry and exit movements associated with each segment. These movements are shown in Exhibit 30-1.

Exhibit 30-1
Entry and Exit Movements on the Typical Street Segment



As indicated in Exhibit 30-1, three entry movements are associated with the upstream signalized intersection and three exit movements are associated with the downstream signalized intersection. Entry and exit movements also exist at each access point intersection. However, these movements are aggregated into one entry and one exit movement for simplicity.

The analysis procedure is described in the following steps. Frequent reference is made to “volume” in these steps. In this application, volume is considered to be equivalent to average flow rate for the analysis period and to have units of vehicles per hour (veh/h).

Step 1: Identify Entry and Exit Volumes

The volume for each entry and exit movement is identified during this step. The volume entering the segment from each access point intersection should be identified and added to obtain a total for the segment. Similarly, the volume exiting the segment from each access point intersection should be identified and added for the segment.

A maximum of eight entry volumes are identified in this step. The seven volumes at the upstream boundary intersection include signalized left-turn volume, signalized through volume, signalized right-turn volume, unsignalized left-turn volume, unsignalized through volume, unsignalized right-turn volume, and right-turn-on-red (RTOR) volume. The eighth entry volume is the total access point entry volume.

A maximum of eight exit volumes are identified in this step. The seven volumes at the downstream boundary intersection include signalized left-turn volume, signalized through volume, signalized right-turn volume, unsignalized left-turn volume, unsignalized through volume, unsignalized right-turn volume, and RTOR volume. The eighth exit volume is the total access point exit volume.

Step 2: Estimate Movement Capacity

During this step, the capacity of each signalized entry movement is estimated. This estimate should be a reasonable approximation based on estimates of the saturation flow rate for the corresponding movement and the phase splits established for signal coordination. The capacity of the RTOR movements is not calculated during this step.

If the right-turn movement at the upstream intersection shares a lane with its adjacent through movement, the discharge flow rate for the turn movement can be estimated by using Equation 30-1.

Equation 30-1

$$s_{q|r} = s_{sr}\, P_R$$

where

$s_{q|r}$ = shared lane discharge flow rate for upstream right-turn traffic movement in vehicles per hour per lane (veh/h/ln),

s_{sr} = saturation flow rate in shared right-turn and through-lane group with permitted operation (veh/h/ln), and

P_R = proportion of right-turning vehicles in the shared lane (decimal).

The procedure described in Section 2 of Chapter 31, Signalized Intersections: Supplemental, is used to estimate the two variables shown in Equation 30-1. A similar equation can be constructed to estimate the shared lane discharge flow rate for an upstream left-turn movement in a shared lane.

The capacity for the right-turn movement in the shared-lane lane group is then computed with Equation 30-2.

Equation 30-2

$$c_{q|r} = s_{q|r}\, g/C$$

where

$c_{q|r}$ = shared lane capacity for upstream right-turn traffic movement (veh/h),

$s_{q|r}$ = shared lane discharge flow rate for upstream right-turn traffic movement (veh/h/ln),

g = effective green time (s), and

C = cycle length (s).

The procedure described in Section 2 of Chapter 31 is used to estimate the signal timing variables shown in Equation 30-2. A similar equation can be constructed for an upstream left-turn movement in a shared lane.

Step 3: Compute Volume-to-Capacity Ratio

During this step, the volume-to-capacity ratio is computed for each signalized entry movement. This ratio is computed by dividing the arrival volume from Step 1 by the capacity estimated in Step 2. Any movements with a volume-to-capacity ratio in excess of 1.0 will meter the volume arriving to the downstream intersection. This ratio is not computed for the RTOR movements.

Step 4: Compute Discharge Volume

The discharge volume from each of the three signalized entry movements is equal to the smaller of its entry volume or its associated movement capacity. The total discharge volume for the combined access point approach is assumed to be equal to the total access point entry volume. Similarly, the discharge volume for each unsignalized and RTOR movement is assumed to equal its corresponding entry volume. As a last calculation, the eight discharge volumes are added to obtain the total discharge volume.

Step 5: Compute Adjusted Exit Volume

The total discharge volume from Step 4 should be compared with the total exit volume. The total exit volume is the sum of the eight exit volumes identified in Step 1. If the two totals do not agree, the eight exit volumes must be adjusted so that their sum equals the total discharge volume. The adjusted exit volume for a movement equals its exit volume multiplied by the "volume ratio." The volume ratio equals the total discharge volume divided by the total exit volume.

Step 6: Repeat Steps 1 Through 5 for Each Segment

The preceding steps should be completed for each segment in the facility in the subject direction of travel. The procedure should then be repeated for the opposing direction of travel.

ORIGIN–DESTINATION DISTRIBUTION

The volume of traffic that arrives at a downstream intersection for a given downstream movement represents the combined volume from each upstream point of entry weighted by its percentage contribution to the downstream exit movement. The distribution of these contribution percentages between each upstream and downstream pair is represented as an origin–destination distribution matrix.

The origin–destination matrix is important for estimating the arrival pattern of vehicles at the downstream intersection. Hence, the focus here is on upstream entry movements that are signalized, because (*a*) they are typically the higher-volume movements and (*b*) the signal timing influences their time of arrival downstream. For these reasons, the origin–destination distribution is focused on the three upstream signalized movements. All other movements (i.e., unsignalized movements at the boundary intersections, access point movements, RTOR movements) are combined into one equivalent movement—referred to hereafter as the "access point" movement—that is assumed to arrive uniformly throughout the signal cycle.

Ideally, an origin–destination survey would be conducted for an existing segment, or the origin–destination data would be available from traffic forecasts by planning models. One matrix would be available for each direction of travel on the segment. In the absence of such information, origin–destination volumes can be estimated from the entry and exit volumes for a segment, where the exit volumes equal the adjusted arrival volumes from the procedure described in the previous subsection, Capacity Constraint and Volume Balance.

Each of the four entry movements to the segment shown in Exhibit 30-1 is considered an origin. Each of the four exit movements is a destination. The problem then becomes one of estimating the origin–destination table given the entering and exiting volumes.

This procedure is derived from research (*1*). It is based on the principle that total entry volume is equal to total exit volume. It uses seed proportions to represent the best estimate of the volume distribution. These proportions are refined through implementation of the procedure. It is derived to estimate the most probable origin–destination volumes by minimizing the deviation from the seed percentages while ensuring the equivalence of entry and exit volumes.

The use of seed percentages allows the procedure to adapt the origin–destination volume estimates to factors or geometric situations that induce greater preference for some entry–exit combinations than is suggested by simple volume proportion (e.g., a downstream freeway on-ramp). The default seed proportions are listed in Exhibit 30-2.

Exhibit 30-2
Default Seed Proportions for Origin–Destination Matrix

Seed Proportion by Origin Movement				Destination Movement
Left	Through	Right	Access Point	
0.02	0.10	0.05	0.02	Left
0.91	0.78	0.92	0.97	Through
0.05	0.10	0.02	0.01	Right
0.02	0.02	0.01	0.00	Access point
1.00	1.00	1.00	1.00	

Step 1: Set Adjusted Origin Volume

$$O_{a,i} = O_i$$

Equation 30-3

where

$O_{a,i}$ = adjusted volume for origin i (i = 1, 2, 3, 4) (veh/h), and

O_i = volume for origin i (i = 1, 2, 3, 4) (veh/h).

The letter i denotes the four movements entering the segment. This volume is computed for each of the four origins.

Step 2: Compute Adjusted Destination Volume

Equation 30-4

$$D_{a,j} = \sum_{i=1}^{4} O_{a,i}\, p_{i,j}$$

where

$D_{a,j}$ = adjusted volume for destination j (j = 1, 2, 3, 4) (veh/h),

$O_{a,i}$ = adjusted volume for origin i (i = 1, 2, 3, 4) (veh/h), and

$p_{i,j}$ = seed proportion of volume from origin i to destination j (decimal).

The letter j denotes the four movements exiting the segment. This volume is computed for each of the four destinations.

Step 3: Compute Destination Adjustment Factor

Equation 30-5

$$b_{d,j} = \frac{D_j}{D_{a,j}}$$

where

$b_{d,j}$ = destination adjustment factor j (j = 1, 2, 3, 4),

D_j = volume for destination j (j = 1, 2, 3, 4) (veh/h), and

$D_{a,j}$ = adjusted volume for destination j (j = 1, 2, 3, 4) (veh/h).

This factor is computed for each of the four destinations.

Step 4: Compute Origin Adjustment Factor

Equation 30-6

$$b_{o,i} = \sum_{j=1}^{4} b_{d,j}\, p_{i,j}$$

where $b_{o,i}$ is the origin adjustment factor i (i = 1, 2, 3, 4). This factor is computed for each of the four origins.

Step 5: Compute Adjusted Origin Volume

Equation 30-7

$$O_{a,i} = \frac{O_i}{b_{o,i}}$$

where $O_{a,i}$ is the adjusted volume for origin i (i = 1, 2, 3, 4) (veh/h). This volume is computed for each of the four origins. It replaces the value previously determined for this variable.

For each origin, compute the absolute difference between the adjusted origin volume from Equation 30-7 and the previous estimate of the adjusted origin volume. If the sum of these four differences is less than 0.01, proceed to Step 6; otherwise, set the adjusted origin volume for each origin equal to the value from Equation 30-7, go to Step 2, and repeat the calculation sequence.

Step 6: Compute Origin–Destination Volume

Equation 30-8

$$v_{i,j} = O_{a,i}\, b_{d,j}\, p_{i,j}$$

where $v_{i,j}$ is the volume entering from origin i and exiting at destination j (veh/h). This volume is computed for all 16 origin–destination pairs.

3. SIGNALIZED SEGMENT ANALYSIS

This section describes the process for analyzing vehicular traffic flow on a segment bounded by signalized intersections. Initially, this process computes the flow profile of discharging vehicles at the upstream intersection as influenced by the signal timing and phase sequence. It uses this profile to compute the arrival flow profile at a downstream junction. The arrival flow profile is then compared with the downstream signal timing and phase sequence to compute the proportion of vehicles arriving during green. The arrival flow profile is also used to compute the proportion of time that a platoon blocks one or more traffic movements at a downstream access point intersection. These two platoon descriptors are used in subsequent procedures to compute delay and other performance measures.

This section describes six procedures that are used to define the arrival flow profile and compute the related platoon descriptors. These procedures are

- Discharge flow profile,
- Running time,
- Projected arrival flow profile,
- Proportion of time blocked,
- Sustained spillback, and
- Midsegment lane restriction.

Each procedure is described in the following subsections.

DISCHARGE FLOW PROFILE

A flow profile is a macroscopic representation of steady traffic flow conditions for the average signal cycle during the specified analysis period. The cycle is represented as a series of 1-s time intervals (hereafter referred to as “time steps”). The start time of the cycle is 0.0 s, relative to the system reference time. The time steps are numbered from 1 to C', where C' is the cycle length in units of time steps. The flow rate for step i represents an average of the flows that occur during the time period corresponding to step i for all cycles in the analysis period. This approach is conceptually the same as that used in the TRANSYT-7F model (*2*).

A discharge flow profile is computed for each of the upstream signalized left-turn, through, and right-turn movements. Each profile is defined by the time that the signal is effectively green and by the time that the queue service time ends. During the queue service time, the discharge flow rate is equal to the saturation flow rate. After the queue service time is reached, the discharge rate is set equal to the “adjusted discharge volume.” The adjusted discharge volume is equal to the discharge volume computed by using the procedures described in Section 2, but it is adjusted to reflect the “proportion of arrivals during green.” The latter adjustment adapts the discharge flow pattern to reflect platoon arrivals on the upstream segment.

The discharge flow profile is dependent on movement saturation flow rate, queue service time, phase duration, and proportion of arrivals during green for the discharging movements. The movement saturation flow rate is computed by using the procedure described in Section 3 of Chapter 19, Signalized Intersections. Procedures for calculating the remaining variables are described in subsequent subsections. This relationship introduces a circularity in the computations that requires an iterative sequence of calculations to converge on the steady-state solution.

RUNNING TIME

The running time procedure describes the calculation of running time between the upstream intersection and a downstream intersection. This procedure is described as Step 2 of the motorized vehicle methodology in Chapter 18, Urban Street Segments.

One component of running time is the delay due to various midsegment sources. One notable source of delay is left or right turns from the segment at an access point intersection. This delay is computed by using the procedure described in Section 4. Other sources of delay include on-street parking maneuvers and pedestrian crosswalks. Delay from these sources represents an input variable to the methodology.

PROJECTED ARRIVAL FLOW PROFILE

This subsection describes the procedure for predicting the arrival flow profile at a downstream intersection (i.e., access point or boundary intersection). This flow profile is based on the discharge flow profile and running time computed previously. The discharge flow profile is used with a platoon dispersion model to compute the arrival flow profile. The platoon dispersion model is summarized in the next part of this subsection. The procedure for using this model to estimate the arrival flow profile is described in the second part.

Platoon Dispersion Model

The platoon dispersion model was originally developed for use in the TRANSYT model (*3*). Input to the model is the discharge flow profile for a specified traffic movement. Output statistics from the model include (*a*) the arrival time of the leading vehicles in the platoon to a specified downstream intersection and (*b*) the flow rate during each subsequent time step.

In general, the arrival flow profile has a lower peak flow rate than the discharge flow profile owing to the dispersion of the platoon as it travels down the street. For similar reasons, the arrival flow profile is spread out over a longer period of time than the discharge flow profile. The rate of dispersion increases with increasing segment running time, which may be caused by access point activity, on-street parking maneuvers, and other midsegment delay sources.

The platoon dispersion model is described by Equation 30-9.

Equation 30-9

$$q'_{a|u,j} = F\, q'_{u,i} + (1 - F)\, q'_{a|u,j-1}$$

with

Equation 30-10

$$j = i + t'$$

where

$q'_{a|u,j}$ = arrival flow rate in time step j at a downstream intersection from upstream source u (veh/step),

$q'_{u,i}$ = departure flow rate in time step i at upstream source u (veh/step),

F = smoothing factor,

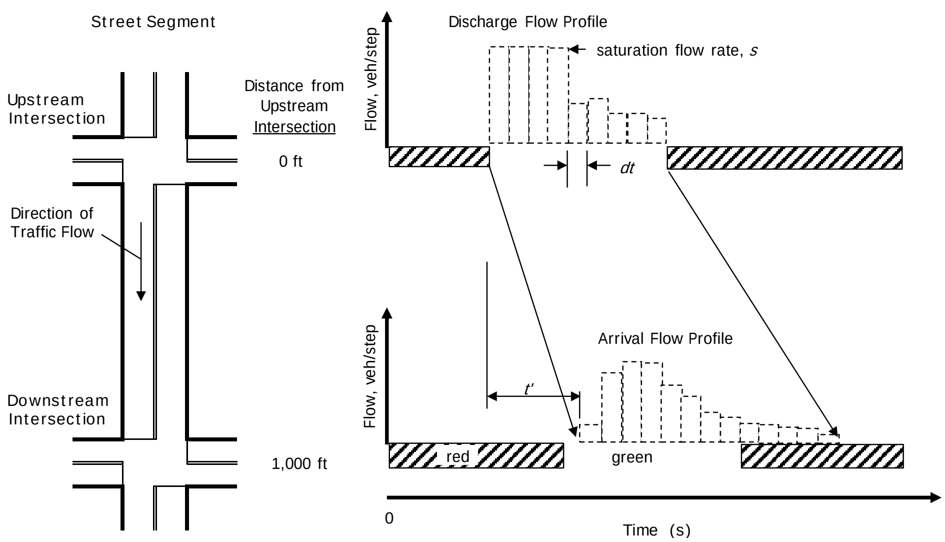
j = time step associated with platoon arrival time t', and

t' = platoon arrival time (steps).

The upstream flow source u can be the left-turn, through, or right-turn movement at the upstream boundary intersection. It can also be the collective set of left-turn or right-turn movements at access point intersections between the upstream boundary intersection and the subject intersection.

Exhibit 30-3 illustrates an arrival flow profile obtained from Equation 30-9. In this figure, the discharge flow profile is input to the model as variable $q'_{u,i}$. The dashed rectangles that form the discharge flow profile indicate the flow rate during each of nine time steps (i = 1, 2, 3, . . . , 9) that are each d_t seconds in duration. The vehicles that depart in the first time step (i = 1) arrive at the downstream intersection after traveling an amount of time equal to t' steps. The arrival flow at any time step j (= i + t') is computed with Equation 30-9.

Exhibit 30-3
Platoon Dispersion Model



Research (*4*) indicates that Equation 30-11 describes the relationship between the smoothing factor and running time.

Equation 30-11

$$F = \frac{1}{1 + 0.138\, t'_R + 0.315/d_t}$$

where

t'_R = segment running time = t_R/d_t (steps),

t_R = segment running time (s), and

d_t = time step duration (s/step).

The recommended time step duration for this procedure is 1.0 s/step. Shorter values can be rationalized to provide a more accurate representation of the profile, but they also increase the time required for the computations. Experience indicates that 1.0 s/step provides a good balance between accuracy and computation time.

Equation 30-12 is used to compute platoon arrival time to the subject downstream intersection.

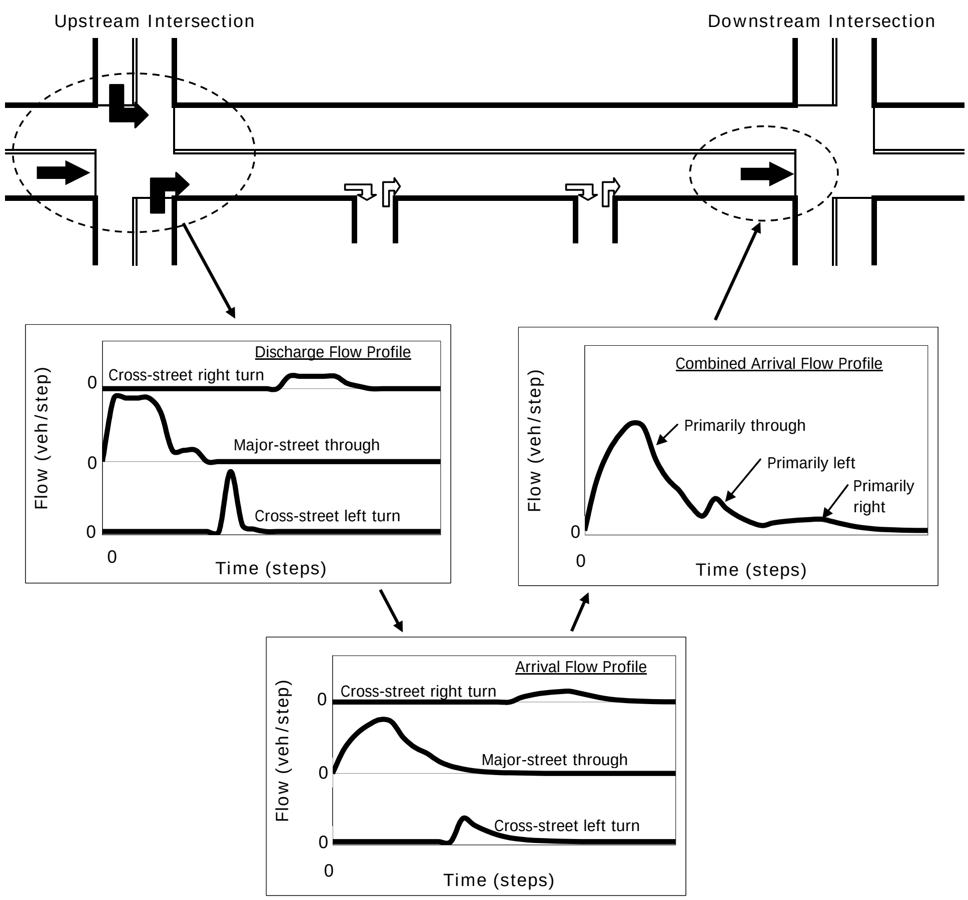
Equation 30-12

$$t' = t'_R - \frac{1}{F} + 1.25$$

Arrival Flow Profile

This subsection describes the procedure for computing the arrival flow profile. Typically, there are three upstream signalized traffic movements that depart at different times during the signal cycle; they are the minor-street right turn, major-street through, and minor-street left turn. Traffic may also enter the segment at various midblock access points or as an unsignalized movement at the boundary intersection. Exhibit 30-4 illustrates how these movements join to form the arrival flow profile for the subject downstream intersection.

Exhibit 30-4
Arrival Flow Profile Estimation Procedure



In application, the discharge flow profile for each of the departing movements is obtained from the discharge flow profile procedure described previously. These profiles are shown in the first of the three x-y plots in Exhibit

30-4. The platoon dispersion model is then used to estimate the arrival flows for each movement at a downstream intersection. These arrival flow profiles are shown in the second *x-y* plot in the exhibit. Arrivals from midsegment access points, which are not shown, are assumed to have a uniform arrival flow profile (i.e., a constant flow rate for all time steps).

Finally, the origin–destination distribution procedure is used to distribute each arrival flow profile to each of the downstream exit movements. The four arrival flow profiles associated with the subject exit movement are added together to produce the combined arrival flow profile. This profile is shown in the third *x-y* plot. The upstream movement contributions to this profile are indicated by arrows.

Comparison of the profiles in the first and second *x-y* plots of Exhibit 30-4 illustrates the platoon dispersion process. In the first *x-y* plot, the major-street through movement has formed a dense platoon as it departs the upstream intersection. However, by the time this platoon reaches the downstream intersection it has spread out and has a lower peak flow rate. In general, the amount of platoon dispersion increases with increasing segment length. For very long segments, the platoon structure degrades and arrivals become uniform throughout the cycle.

Platoon structure can also degrade as a result of significant access point activity along the segment. Streets with frequent active access point intersections tend to have more vehicles leave the platoon (i.e., turn from the segment at an access point) and enter the segment after the platoon passes (i.e., turn in to the segment at an access point). Both activities result in significant platoon decay.

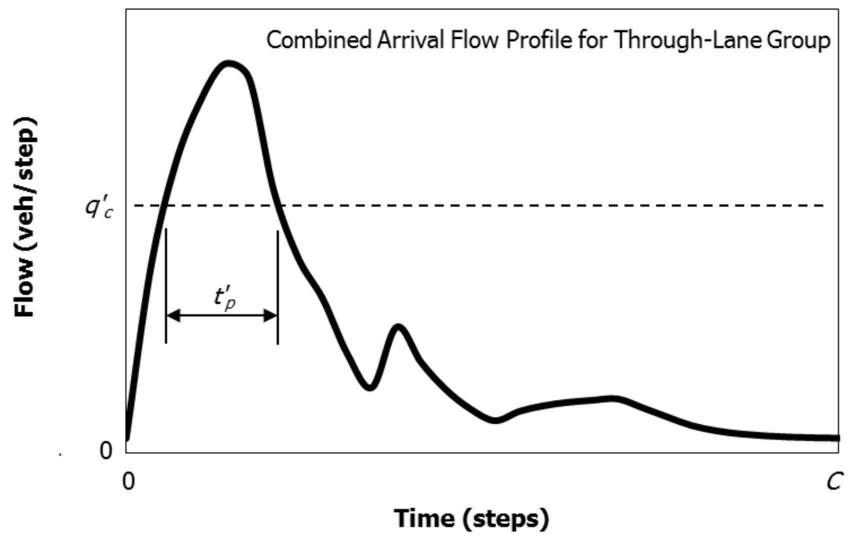
The effect of platoon decay is modeled by using the origin–destination matrix, in which the combined access point activity is represented as one volume assigned to midsegment origins and destinations. A large access point volume corresponds to a smaller volume that enters at the upstream boundary intersection as a defined platoon. This results in a larger portion of the combined arrival flow profile defined by uniform (rather than platoon) arrivals. When a street has busy access points, platoon decay tends to be a more dominant cause of platoon degradation than platoon dispersion.

PROPORTION OF TIME BLOCKED

The combined arrival flow profile can be used to estimate the time that a platoon passes through a downstream access point intersection. During this time period, the platoon can be dense enough to preclude a minor movement driver from finding an acceptable gap.

The use of the arrival flow profile to estimate the blocked period duration is shown in Exhibit 30-5. The profile shown represents the combined arrival flow profile for the through-lane group at a downstream access point intersection. The dashed line represents the critical platoon flow rate. Flow rates in excess of this threshold are rationalized to be associated with platoon headways that are too short to be entered (or crossed) by minor movements. The critical platoon flow rate q_c is equal to the inverse of the critical headway t_c associated with the minor movement (i.e., $q_c = 3{,}600/t_c$). The appropriate critical headway values for various movements are identified in Chapter 20, Two-Way STOP-Controlled Intersections.

Exhibit 30-5
Estimation of Blocked Period Duration



In the situation of a driver desiring to complete a left turn from the major street across the traffic stream represented by Exhibit 30-5, the proportion of time blocked is computed by using Equation 30-13. For this maneuver, the blocked period duration is based on the flow profile of the opposing through-lane group.

Equation 30-13

$$p_b = \frac{t'_p \, d_t}{C}$$

where

p_b = proportion of time blocked (decimal),

t'_p = blocked period duration (steps),

d_t = time step duration (s/step), and

C = cycle length (s).

Equation 30-13 is also used for the minor-street right-turn movement. However, in this situation, the blocked period duration is computed for the through-lane group approaching from the left. For the minor-street left-turn and through movements, the arrival flow profiles from both directions are evaluated. In this instance, the blocked period duration represents the time when a platoon from either direction is present in the intersection.

SUSTAINED SPILLBACK

This subsection describes two procedures that were developed for the evaluation of segments that experience sustained spillback. Sustained spillback occurs as a result of oversaturation (i.e., more vehicles discharging from the upstream intersection than can be served at the subject downstream intersection). The spillback can exist at the start of the study period, or it can occur at some point during the study period. Spillback that first occurs after the study period is not addressed.

Effective Average Vehicle Spacing

One piece of information needed to evaluate segments experiencing sustained spillback is the effective average vehicle spacing (*5*). A simple estimate of this spacing is computed as the sum of the average vehicle length and the average distance between two queued vehicles (as measured from the back bumper of the lead vehicle to the front bumper of the trailing vehicle).

Presumably, this estimate of average spacing could be divided into the segment length to determine the maximum number of queued vehicles on the segment during spillback. However, this result is biased because it is based on the assumption that all vehicles on the segment will always be stationary during spillback. This is a weak assumption because the downstream signal operation creates backward-traveling waves of starting and stopping. Between the starting wave and the stopping wave, vehicles are moving at the saturation headway and its associated speed. Their spacing exceeds that of the aforementioned "simple" estimate.

The procedure described in this subsection is used to estimate the effective average vehicle spacing L_h^* on a segment with spillback. The derivation of this new variable is based on the vehicle trajectories shown in Exhibit 30-6. The segment of interest is shown on the left side of the figure. Spillback is present for all of the cycles shown; however, trajectories are shown only for two cycles. The solid trajectories coincide with vehicles that enter the segment as a through movement at the upstream intersection. The dashed lines coincide with vehicles that enter the segment as a turn movement. A vehicle that enters the segment traveling north as a through vehicle is shown to experience four cycles before exiting the segment. The trajectories show that the vehicles move forward at a saturation headway of 3,600/*s* seconds per vehicle and a speed of V_a feet per second.

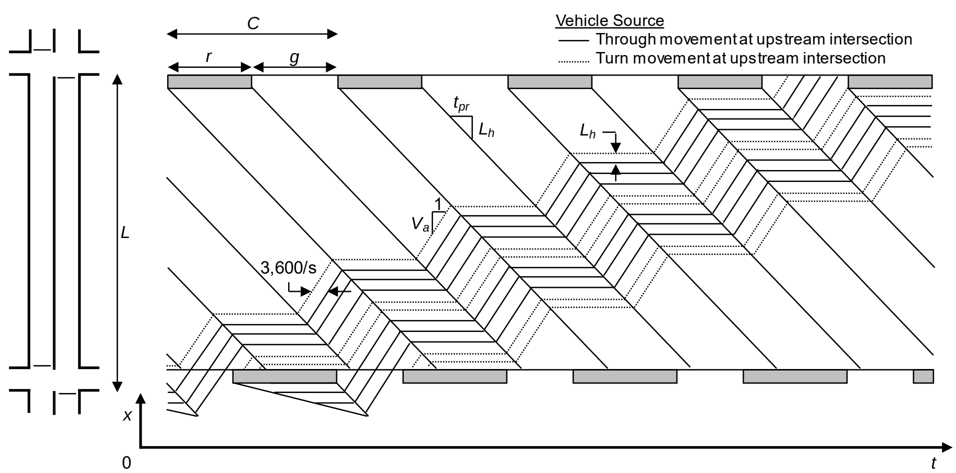


Exhibit 30-6
Vehicle Trajectories During Spillback Conditions

The lines that slope downward from the upper left to lower right represent the waves of reaction time. They have a slope of t_{pr} seconds per vehicle. The starting wave originates at the onset of the green indication, and the stopping wave originates at the onset of the red indication. The average vehicle spacing when vehicles are stopped is L_h feet per vehicle.

On the basis of the relationships shown in Exhibit 30-6, the following procedure can be used to estimate the effective average vehicle spacing.

Step 1. Compute Wave Travel Time

The time required for the driver reaction wave to propagate backward to the upstream intersection is computed with the following equation:

Equation 30-14

$$t_{max} = \frac{(L - W_i)\, t_{pr}}{L_h}$$

with

Equation 30-15

$$L_h = L_{pc}(1 - 0.01\, P_{HV}) + 0.01\, L_{HV}\, P_{HV}$$

where

t_{max} = wave travel time (s);

L = segment length (ft);

W_i = width of upstream signalized intersection, as measured along the segment centerline (ft);

t_{pr} = driver starting response time (= 1.3) (s/veh);

L_h = average vehicle spacing in stationary queue (ft/veh);

L_{pc} = stored passenger car lane length = 25 (ft);

L_{HV} = stored heavy vehicle lane length = 45 (ft); and

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

Step 2. Compute Speed of Moving Queue

The average speed of the moving queue is computed with Equation 30-16:

Equation 30-16

$$V_a = \frac{L_h}{2.0 - t_{pr}}$$

where V_a is the average speed of moving queue (ft/s).

Step 3. Compute Effective Average Vehicle Spacing

The relationship between the trajectories of the moving vehicles defines the following association between speed, saturation flow rate, signal timing, and vehicle spacing.

Equation 30-17

$$\text{If } 0.0 \le t_{max} < r, \text{ then } L_h^* = L_h$$

$$\text{If } r \le t_{max} < C, \text{ then } L_h^* = 2.0\left(\frac{r}{L - W_i} + \frac{1}{V_a}\right)^{-1} \ge L_h$$

$$\text{If } C \le t_{max}, \text{ then } L_h^* = \frac{L_h}{1.0 - 0.5\, t_{pr}\, g/C}$$

where

L_h^* = effective average vehicle spacing in stationary queue (ft/veh),

r = effective red time (= $C - g$) (s),

g = effective green time (s), and

C = cycle length (s).

Equation 30-17 has three component equations. The component equation used for a given segment and analysis period will be based on the value of t_{max}, r, and C. The value of average vehicle spacing from the first component equation is the smallest that can be obtained from Equation 30-17. The value from the last equation is the largest that can be obtained. The value obtained from the equation in the middle varies between these two extreme values, depending on the value of t_{max}.

Spillback Check

This subsection describes the procedure for determining whether queue spillback occurs on a segment during a given analysis period (*4*). The analysis is applied separately to each travel direction and proceeds in the direction of travel. The procedure consists of a series of steps that are completed in sequence for the signalized exit movements associated with each segment. These movements were shown in Exhibit 30-1. Spillback due to the movements associated with the access points is not specifically addressed.

Step 1: Identify Initial Queue

During this step, the initial queue for each signalized exit movement is identified. This value represents the queue present at the start of the analysis period (the total of all vehicles in all lanes serving the movement). The initial queue estimate would likely be available for the evaluation of an existing condition for which field observations indicate the presence of a queue at the start of the analysis period. For planning or preliminary design applications, it can be assumed to equal 0.0 vehicles.

Step 2: Identify Queue Storage Length

The length of queue storage for each exit movement is identified during this step. For turn movements served from a turn bay, this length equals the length of the turn bay. For through movements, this length equals the segment length less the width of the upstream intersection. For turn movements served from a lane equal in length to that of the segment, the queue storage length equals the segment length less the width of the upstream intersection.

Step 3: Compute Maximum Queue Storage

The maximum queue storage for the exiting through movement is computed with Equation 30-18:

$$N_{qx,thru} = \frac{(N_{th} - P_L - P_R)\, L_{a,thru}}{L_h^*}$$

Equation 30-18

where

$N_{qx,thru}$ = maximum queue storage for the through movement (veh),

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

P_L = proportion of left-turning vehicles in the shared lane (decimal),

P_R = proportion of right-turning vehicles in the shared lane (decimal),

$L_{a,thru}$ = available queue storage distance for the through movement (ft/ln), and

L_h^* = effective average vehicle spacing in stationary queue (ft/veh).

The procedure described in Section 2 of Chapter 31, Signalized Intersections: Supplemental, is used to estimate P_L and P_R. If there are no shared lanes, $P_L = 0.0$ and $P_R = 0.0$.

The maximum queue storage for a turn movement is computed with Equation 30-19:

Equation 30-19

$$N_{qx,\text{turn}} = \frac{N_{\text{turn}}\, L_{a,\text{turn}} + P_{\text{turn}}\, L_{a,thru}}{L_h}$$

where

$N_{qx,\text{turn}}$ = maximum queue storage for a turn movement (veh),

N_{turn} = number of lanes in the turn bay (ln),

$L_{a,\text{turn}}$ = available queue storage distance for the turn movement (ft/ln),

P_{turn} = proportion of turning vehicles in the shared lane = P_L or P_R (decimal), and

L_h = average vehicle spacing in stationary queue (ft/veh).

This equation is applicable to turn movements in exclusive lanes (i.e., $P_{\text{turn}} = 0.0$) and to turn movements that share a through lane.

Step 4: Compute Available Storage Length

The available storage length is computed for each signalized exit movement by using Equation 30-20.

Equation 30-20

$$N_{qa} = N_{qx} - Q_b \geq 0.0$$

where

N_{qa} = available queue storage (veh),

N_{qx} = maximum queue storage for the movement (veh), and

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

The analysis thus far has treated the three signalized exit movements as if they were independent. At this point, the analysis must be extended to include the combined through and left-turn movement when the left-turn movement has a bay (i.e., it does not have a lane that extends the length of the segment). The analysis must also be extended to include the combined through and right-turn movement when the right-turn movement has a bay (but not a full-length lane).

The analysis of these newly formed “combined movements” is separated into two parts. The first part is the analysis of just the bay. This analysis is a continuation of the exit movement analysis using the subsequent steps of this procedure. The second part is the analysis of the length of the segment shared by the turn movement and the adjacent through movement. The following rules are used to evaluate the combined movements for the shared segment length:

1. The volume for each combined movement equals the sum of the adjusted arrival volumes for the two contributing movements. These volumes are

obtained from the procedure described in a previous subsection, Origin–Destination Distribution.

2. The initial queue for each combined movement is computed with Equation 30-21.

$$Q_{b,comb} = \max\left(0.0, Q_{b,\text{turn}} - \frac{L_{a,\text{turn}}\, N_{\text{turn}}}{L_h^*}, Q_{b,thru} - \frac{L_{a,\text{turn}}\, N_{th}}{L_h^*}\right)$$

Equation 30-21

where $Q_{b,comb}$ is the initial queue for the combined movement (veh). The other variables were defined previously and are evaluated for the movement indicated by the variable subscript.

3. The queue storage length for a combined movement $L_{a,comb}$ equals the queue storage length for the through movement less the queue storage length of the turn movement (i.e., $L_{a,comb} = L_{a,thru} - L_{a,\text{turn}}$).

4. The number of lanes available to the combined movement N_{comb} equals the number of lanes available to the through movement.

5. The maximum queue storage for the combined movement $N_{qx,comb}$ is computed with the following equation:

$$N_{qx,comb} = \frac{N_{th}\, L_{a,thru}}{L_h^*}$$

Equation 30-22

6. The available storage length for the combined movement $N_{qa,comb}$ is computed with the following equation:

$$N_{qa,comb} = N_{qx,comb} - Q_{b,comb} \geq 0.0$$

Equation 30-23

Step 5: Compute Capacity

The capacity for both the exit movements and the combined movements is established in this step. The capacity for each exit movement was computed in Step 2 in the subsection titled Capacity Constraint and Volume Balance. The capacity of the combined movements is computed by using Equation 30-24.

$$c = \frac{v_{a,1}}{X_1} + \frac{c_{thru}(N_{th} - 1)}{N_{th}}$$

Equation 30-24

with

$$v_{a,1} = \max\left(v_{a,\text{turn}}, \frac{v_{a,\text{turn}} + v_{a,thru}}{N_{th}}\right)$$

Equation 30-25

$$X_1 = \frac{v_{a,\text{turn}}}{c_{\text{turn}}} + \frac{v_{a,1} - v_{a,\text{turn}}}{c_{thru}/N_{th}}$$

Equation 30-26

where

c = capacity of the combined movements (veh/h),

$v_{a,1}$ = adjusted arrival volume in the shared lane (veh/h),

X_1 = volume-to-capacity ratio in the shared lane,

c_{thru} = capacity for the exiting through movement (veh/h),

c_{turn} = capacity for the exiting turn movement (veh/h),

$v_{a,\text{turn}}$ = adjusted arrival volume for the subject turn movement (veh/h),

$v_{a,thru}$ = adjusted arrival volume for the subject through movement (veh/h), and

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

The two adjusted arrival volumes $v_{a,\text{turn}}$ and $v_{a,thru}$ are obtained from the procedure described in the Origin–Destination Distribution subsection.

Step 6: Compute Queue Growth Rate

During this step, the queue growth rate is computed for each signalized exit movement for which the storage extends the length of the segment. Typically, the through movement satisfies this requirement. A turn movement may also satisfy this requirement if it is served by an exclusive lane that extends the length of the segment. The queue growth rate is computed as the difference between the adjusted arrival volume v_a and the capacity c for the subject exit movement. Equation 30-27 is used to compute this rate.

Equation 30-27

$$r_{qg} = v_a - c \geq 0.0$$

where r_{qg} is the queue growth rate (veh/h).

The queue growth rate is also computed for the combined movements formulated in Step 4. The adjusted volume used in Equation 30-27 represents the sum of the through and turn movement volumes in the combined group. The capacity for the group was computed in Step 5.

Step 7: Compute Time Until Spillback

During this step, the time until spillback is computed for each signalized exit movement for which the storage extends the length of the segment. This time is computed with Equation 30-28 for any movement with a nonzero queue growth rate.

Equation 30-28

$$T_c = \frac{N_{qa}}{r_{qg}}$$

where T_c is the time until spillback (h).

For turn movements served by a bay, the computed spillback time is the time required for the bay to overflow. It does not represent the time at which the turn-related queue reaches the upstream intersection.

Equation 30-28 is also used to compute the spillback time for the combined movements formulated in Step 4. However, this spillback time is the additional time required for the queue to grow along the length of segment shared by the turn movement and the adjacent through movement. This time must be added to the time required for the corresponding turn movement to overflow its bay to obtain the actual spillback time for the combined movement.

Step 8: Repeat Steps 1 Through 7 for Each Segment

The preceding steps should be completed for each segment in the facility in the subject direction of travel. The procedure should then be repeated for the opposing direction of travel.

Step 9: Determine Controlling Spillback Time

During this step, the shortest time until spillback for each of the exit movements (or movement groups) for each segment and direction of travel is identified. If the segment supports two travel directions, two values are identified (one value for each direction). The smaller of the two values is the controlling spillback time for the segment. If a movement (or movement group) does not spill back, it is not considered in this process for determining the controlling spillback time.

Next, the controlling segment times are compared for all segments that make up the facility. The shortest time found is the controlling spillback time for the facility.

If the controlling spillback time exceeds the analysis period, the results from the motorized vehicle methodology are considered to reflect the operation of the facility accurately. If spillback occurs before the end of the desired analysis period, the analyst should consider either (*a*) reducing the analysis period so that it ends before spillback occurs or (*b*) using the sustained spillback evaluation procedure in Chapter 29, Urban Street Facilities: Supplemental.

MIDSEGMENT LANE RESTRICTION

When one or more lanes on an urban street segment are temporarily closed, the flow in the lanes that remain open can be adversely affected. The closure can be due to a work zone, an incident, or a similar event. Occasionally, the lane closure can adversely affect the performance of traffic movements that are entering or exiting the segment at the boundary signalized intersection. Logically, the magnitude of the effect will increase as the distance between the intersection and lane closure decreases. The impact on the intersection that has a downstream lane closure is the subject of discussion in this subsection.

The procedure described in this subsection is used to adjust the saturation flow rate of the movements entering a segment when one or more downstream lanes are blocked. The procedure is developed for incorporation within the motorized vehicle methodology described in Chapters 18 and 19 (*5*). Specifically, the procedure is inserted into the motorized vehicle methodology in Chapter 18, Urban Street Segments, and used to compute a saturation flow rate adjustment factor for the movements entering the segment at the intersection. This adjustment factor is then implemented in the motorized vehicle methodology in Chapter 19, Signalized Intersections, to compute the adjusted saturation flow rate of the affected movements.

This procedure is added to the end of Step 4 of the motorized vehicle methodology described in Chapter 18. It occurs after the saturation flow rate and phase duration have been determined. It is implemented as part of the iterative convergence loop identified in the motorized vehicle methodology framework shown in Exhibit 18-8.

The calculation sequence begins with an estimate of the capacity for each traffic movement discharged to the downstream segment. This estimate is obtained by using the motorized vehicle methodology in Chapter 19. The next step is to compute the capacity of the downstream segment as influenced by the midsegment lane restriction. The estimate of movement capacity is then compared with the downstream segment capacity. If the movement capacity exceeds the downstream segment capacity, the movement saturation flow rate is reduced proportionally by using an adjustment factor for downstream lane blockage.

The lane blockage saturation flow rate adjustment factor is computed for each movement entering the subject segment. The following equations are used to compute the factor value.

Equation 30-29

$$\text{If } c_{ms} < c_i \text{ or } f_{ms,i-1} < 1.0, \text{ then } f_{ms,i} = f_{ms,i-1}\frac{c_{ms}}{c_i} \geq 0.1$$

$$\text{Otherwise, } f_{ms,i} = 1.0$$

with

Equation 30-30

$$c_{ms} = 0.25\, k_j\, N_{unblk}\, S_f \leq 1{,}800\, N_{unblk}$$

where

$f_{ms,i}$ = adjustment factor for downstream lane blockage during iteration i,

c_{ms} = midsegment capacity (veh/h),

c_i = movement capacity during iteration i (veh/h),

k_j = jam density (= 5,280 / L_h) (veh/mi/ln),

L_h = average vehicle spacing in stationary queue (ft/veh),

S_f = free-flow speed (mi/h), and

N_{unblk} = number of open lanes when blockage is present (ln).

The number of lanes used in Equation 30-30 equals the number of unblocked lanes (i.e., the open lanes) while the blockage is present.

The variable i in the adjustment factor subscript indicates that the factor's value is incrementally revised during each iteration of the convergence loop associated with the motorized vehicle methodology. Ultimately, the factor converges to a value that results in a movement capacity matching the available midsegment capacity. For the first iteration, the factor value is set to 1.0 for all movements. The factor value is also set to 1.0 if the segment is experiencing spillback. In this situation, a saturation flow rate adjustment factor for spillback (which incorporates the downstream lane blockage effect) is computed for the movement. The calculation of the factor for spillback is described in Chapter 29, Urban Street Facilities: Supplemental.

Equation 30-29 indicates that the factor is less than 1.0 when the midsegment capacity is smaller than the movement capacity. If the factor has been set to a value less than 1.0 in a previous iteration, it continues to be adjusted during each subsequent iteration until convergence is achieved. A minimum factor value of 0.1 is imposed as a practical lower limit.

4. DELAY DUE TO TURNS

This section describes a process for estimating the delay to through vehicles that follow vehicles turning from the major street into an unsignalized access point intersection. This delay can be incurred at any access point intersection along the street. For right-turn vehicles, the delay results when the following vehicles' speed is reduced to accommodate the turning vehicle. For left-turn vehicles, the delay results when the following vehicles must wait in queue while a vehicle ahead executes a left-turn maneuver at the access point. This delay occurs primarily on undivided streets; however, it can occur on divided streets when the left-turn queue exceeds the available storage and spills back into the inside through lane.

The delay estimation process consists of the following two procedures:

- Delay due to left turns and
- Delay due to right turns.

Each procedure is described in the following subsections. These procedures are based on the assumption that the segment traffic flows are random. While this assumption may not be strictly correct for urban streets, it is conservative in that it will yield slightly larger estimates of delay. Moreover, expansion of the models to accommodate platooned flows would not likely be cost-effective given the small amount of delay caused by turning vehicles.

DELAY DUE TO LEFT TURNS

Through vehicles on the major-street approach to an unsignalized intersection can incur delay when the left-turn queue exceeds the available storage and blocks the adjacent through lane (in this context, the undivided cross section is considered a major-street approach having no left-turn storage). The through vehicles that follow are delayed when they stop behind the queue of turning vehicles. This delay ends when the left-turn vehicle departs or the through vehicle merges into the adjacent through lane. By merging into the adjacent lane, drivers reduce their delay relative to the delay they would have incurred had they waited for the left-turn queue to clear. This delay is computed by using Equation 30-31.

$$d_{ap,l} = p_{ov}\, d_{t,1}\left(\frac{1}{P_L} - 1\right)\frac{P_{lt}}{1 - P_{lt} - P_{rt}}$$

Equation 30-31

where

$d_{ap,l}$ = through-vehicle delay due to left turns (s/veh),

p_{ov} = probability of left-turn bay overflow (decimal),

$d_{t,1}$ = average delay to through vehicles in the inside lane (s/veh),

P_L = proportion of left-turning vehicles in the shared lane (decimal),

P_{lt} = proportion of left-turning vehicles on the subject approach (decimal), and

P_{rt} = proportion of right-turning vehicles on the subject approach (decimal).

As indicated by Equation 30-31, the delay due to left turns is based on the value of several variables. The following sequence of computations can be used to estimate these values (*6*).

Step 1: Compute the Probability of a Lane Change

Equation 30-32

$$P_{lc} = 1 - \left[\left(2\frac{v_{app}}{s_{lc}}\right) - 1\right]^2 \geq 0.0$$

with

Equation 30-33

$$v_{app} = \frac{v_{lt} + v_{th} + v_{rt}}{N_{sl} + N_t + N_{sr}}$$

where

- P_{lc} = probability of a lane change among the approach through lanes,
- v_{app} = average demand flow rate per through lane (upstream of any turn bays on the approach) (veh/h/ln),
- s_{lc} = maximum flow rate in which a lane change can occur = 3,600/t_{lc} (veh/h/ln),
- t_{lc} = critical merge headway = 3.7 (s),
- v_{lt} = left-turn demand flow rate (veh/h),
- v_{th} = through demand flow rate (veh/h),
- v_{rt} = right-turn demand flow rate (veh/h),
- N_{sl} = number of lanes in shared left-turn and through-lane group (ln),
- N_t = number of lanes in exclusive through-lane group (ln), and
- N_{sr} = number of lanes in shared right-turn and through-lane group (ln).

If the ratio v_{app}/s_{lc} in Equation 30-32 exceeds 1.0, then it should be set to 1.0.

Step 2: Compute Through-Vehicle Equivalent for Left-Turn Vehicle

If there is a left-turn bay on the major street at the access point, the through-vehicle equivalent E_{L1} is 1.0. However, if there is no left-turn bay, the following equation is used to compute the through-vehicle equivalent.

Equation 30-34

$$E_{L1} = \frac{1{,}800}{c_l}$$

with

Equation 30-35

$$c_l = \frac{v_o\ e^{-v_o\ t_{cg}/3{,}600}}{1 - e^{-v_o\ t_{fh}/3{,}600}}$$

where

- E_{L1} = equivalent number of through cars for a permitted left-turning vehicle,
- c_l = capacity of a left-turn movement with permitted left-turn operation (veh/h),
- v_o = opposing demand flow rate (veh/h),

t_{fh} = follow-up headway = 2.2 (s), and

t_{cg} = critical headway = 4.1 (s).

Step 3: Compute Modified Through-Vehicle Equivalent

$$E_{L1,m} = (E_{L1} - 1)P_{lc} + 1$$ Equation 30-36

$$E_{R,m} = \left(E_{R,ap} - 1\right)P_{lc} + 1$$ Equation 30-37

where

$E_{L1,m}$ = modified through-car equivalent for a permitted left-turning vehicle,

$E_{R,m}$ = modified through-car equivalent for a protected right-turning vehicle, and

$E_{R,ap}$ = equivalent number of through cars for a protected right-turning vehicle at an access point (2.20 if there is no right-turn bay on the major street at the access point; 1.0 if there is a right-turn bay).

Step 4: Compute Proportion of Left Turns in Inside Through Lane

$$P_L = \frac{-b + \sqrt{b^2 - 4\, I_t\, R\, c}}{2\, I_t\, R} \le 1.0$$ Equation 30-38

with

$$b = R - I_{lt}\, P_{lt}\left\{I_t + (N_{sl} + N_t + N_{sr} - 1)\left[(1 + I_t)E_{L1,m} - 1\right]\right\}$$ Equation 30-39

$$c = -I_{lt}\, P_{lt}(N_{sl} + N_t + N_{sr})$$ Equation 30-40

$$R = 1 + I_{rt}\, P_{rt}(E_{R,m} - 1)$$ Equation 30-41

where

R, b, c = intermediate calculation variables;

I_{lt} = indicator variable (1.0 when there is no left-turn bay on the major street at the access point, 0.0 when there is a left-turn bay);

I_{rt} = indicator variable (1.0 when there is no right-turn bay on the major street at the access point, 0.0 when there is a right-turn bay); and

I_t = indicator variable (1.0 when equations are used to evaluate delay due to left turns, 0.00001 when equations are used to evaluate delay due to right turns).

If the number of through lanes on the subject intersection approach (= $N_{sl} + N_t + N_{sr}$) is equal to 1.0, then $P_L = P_{lt}$.

The indicator variable I_t is used to adapt the equations to the analysis of lane volume for both left-turn- and right-turn-related delays. The variable has a value of 1.0 in the evaluation of left-turn-related delays. In this situation, it models the condition in which one or more left-turning vehicles are blocking the inside lane. In contrast, the variable has a negligibly small value when it is applied to right-turn-related delays. It models flow conditions in which all lanes are unblocked.

Step 5: Compute Proportion of Right Turns in Outside Through Lane

Equation 30-42

$$P_R = I_{rt}\, P_{rt} \frac{\frac{s_1}{1{,}800} + N_{sl} + N_t + N_{sr} - 1}{1 - I_{rt}\, P_{rt}\left(\frac{s_1}{1{,}800} + N_{sl} + N_t + N_{sr} - 2\right)\left(E_{R,m} - 1\right)} \leq 1.0$$

with

Equation 30-43

$$s_1 = \frac{1{,}800\,(1 + P_L\, I_t)}{1 + P_L\left(E_{L1,m} - 1\right) + (P_L\, E_{L1,m}\, I_t)}$$

where s_1 is the saturation flow rate for the inside lane (veh/h/ln). If the number of through lanes on the subject intersection approach (= $N_{sl} + N_t + N_{sr}$) is equal to 1.0, then $P_R = P_{rt}$.

Step 6: Compute Inside Lane and Outside Lane Flow Rates

Equation 30-44

$$v_1 = \frac{v_{lt}}{P_L}$$

Equation 30-45

$$v_n = \begin{cases} \dfrac{v_{rt}}{P_R} & \text{if } P_R > 0.0 \\[2ex] \dfrac{v_{lt} + v_{th} + v_{rt} - v_1}{N_{sl} + N_t + N_{sr} - 1} & \text{if } P_R = 0.0 \end{cases}$$

where

v_1 = flow rate for the inside lane (veh/h/ln) and

v_n = flow rate for the outside lane (veh/h/ln).

Step 7: Compute Intermediate Lane Flow Rate

If there are more than two lanes on the subject intersection approach, Equation 30-46 can be used to estimate the flow rate in the intermediate lanes.

Equation 30-46

$$v_i = \frac{v_{lt} + v_{th} + v_{rt} - v_1 - v_n}{N_{sl} + N_t + N_{sr} - 2}$$

where v_i is the flow rate for lane i (veh/h/ln). The flow rates in lanes 2, 3, . . . , $n - 1$ are identical and equal to the value obtained from Equation 30-46.

Step 8: Compute Merge Capacity

Equation 30-47 is used to compute the merge capacity available to through drivers waiting in the inside lane of a multilane approach.

Equation 30-47

$$c_{mg} = \frac{v_2\, e^{-v_2\, t_{lc}/3{,}600}}{1 - e^{-v_2\, t_{lc}/3{,}600}}$$

where

c_{mg} = merge capacity (veh/h),

v_2 = flow rate in the adjacent through lane (veh/h/ln), and

t_{lc} = critical merge headway = 3.7 (s).

Step 9: Compute Delay to Through Vehicles That Merge

$$d_{mg} = 3{,}600\left(\frac{1}{c_{mg}} - \frac{1}{1{,}800}\right) + 900\,T\left[\frac{v_{mg}}{c_{mg}} - 1 + \sqrt{\left(\frac{v_{mg}}{c_{mg}} - 1\right)^2 + \frac{8\,v_{mg}}{c_{mg}^2\,T}}\right]$$

Equation 30-48

with

$$v_{mg} = v_1 - v_{lt} \geq 0.0$$

Equation 30-49

where

d_{mg} = merge delay (s/veh),

v_{mg} = merge flow rate (veh/h/ln), and

T = analysis period duration (h).

This delay is incurred by through vehicles that stop in the inside lane and eventually merge into the adjacent through lane. The "1/1,800" term included in Equation 30-48 extracts the service time for the through vehicle from the delay estimate, so that the delay estimate represents the increase in travel time resulting from the left-turn queue.

Step 10: Compute Inside Lane Capacity

Equation 30-50 is used to compute the capacity of the inside lane for vehicles that do not merge.

$$c_{nm} = \frac{1{,}800(1 + P_L)}{1 + P_L(E_{L1} - 1) + (P_L E_{L1})}$$

Equation 30-50

where c_{nm} is the nonmerge capacity for the inside lane (veh/h). The unadjusted through-vehicle equivalent for a left-turn vehicle E_{L1} is used in this equation to estimate the nonmerge capacity.

Step 11: Compute Delay to Through Vehicles That Do Not Merge

$$d_{nm} = 3{,}600\left(\frac{1}{c_{nm}} - \frac{1}{1{,}800}\right) + 900\,T\left[\frac{v_1}{c_{nm}} - 1 + \sqrt{\left(\frac{v_1}{c_{nm}} - 1\right)^2 + \frac{8\,v_1}{c_{nm}^2\,T}}\right]$$

Equation 30-51

where d_{nm} is the nonmerge delay for the inside lane (s/veh). This delay is incurred by through vehicles that stop in the inside lane and wait for the queue to clear. These vehicles do not merge into the adjacent lane.

Step 12: Compute Delay to Through Vehicles in the Inside Lane

This delay is estimated as the smaller of the delay relating to the merge and nonmerge maneuvers. It is computed with Equation 30-52.

$$d_{t,1} = \min\left(d_{nm}, d_{mg}\right)$$

Equation 30-52

Step 13: Compute the Probability of Left-Turn Bay Overflow

The probability of left-turn bay overflow is computed by using the following equation:

$$p_{ov} = \left(\frac{v_{lt}}{c_l}\right)^{N_{qx,lt}+1}$$

Equation 30-53

with

Equation 30-54

$$N_{qx,lt} = \frac{N_{lt}\, L_{a,lt}}{L_h}$$

where

p_{ov} = probability of left-turn bay overflow (decimal),

$N_{qx,lt}$ = maximum queue storage for the left-turn movement (veh),

N_{lt} = number of lanes in the left-turn bay (ln),

$L_{a,lt}$ = available queue storage distance for the left-turn movement (ft/ln), and

L_h = average vehicle spacing in the stationary queue (see Equation 30-15) (ft/veh).

For an undivided cross section, the number of left-turn vehicles that can be stored, $N_{qx,lt}$, is equal to 0.0.

Step 14: Compute Through-Vehicle Delay due to Left Turns

The through-vehicle delay due to left turns $d_{ap,l}$ is computed with Equation 30-31.

DELAY DUE TO RIGHT TURNS

A vehicle turning right from the major street into an access point often delays the through vehicles that follow it. Through vehicles are delayed because they have to reduce speed to avoid a collision with the vehicle ahead, the first of which has reduced speed to avoid a collision with the right-turning vehicle. This delay can be several seconds in duration for the first few through vehicles but will always decrease to negligible values for subsequent vehicles as the need to reduce speed diminishes. For purposes of running time calculation, this delay must be averaged over all through vehicles traveling in the subject direction. The resulting average delay is computed with Equation 30-55.

Equation 30-55

$$d_{ap,r} = 0.67\, d_{t|r} \frac{P_{rt}}{1 - P_{lt} - P_{rt}}$$

where

$d_{ap,r}$ = through-vehicle delay due to right turns (s/veh) and

$d_{t|r}$ = through-vehicle delay per right-turn maneuver (s/veh).

The variable $d_{t|r}$ in Equation 30-55 converges to 0.0 as the proportion of turning vehicles approaches 1.0. The constant 0.67 is a calibration factor based on field data. The steps undertaken to quantify this factor are described in the remainder of this subsection. Equation 30-55 can also be used to estimate the delay due to left-turn vehicles on a one-way street. In this case, variables associated with the right-turn movement would be redefined as applicable to the left-turn movement and vice versa.

As indicated by Equation 30-55, the delay due to right turns is based on the value of several variables. The following sequence of computations can be used to estimate these values (*7*).

Step 1: Compute Minimum Speed for the First Through Vehicle

$$u_m = 1.47\, S_f - r_d(H_1 - h_{|\Delta<h<H_1}) \geq u_{rt}$$ Equation 30-56

with

$$h_{|\Delta<h<H_1} = \frac{1}{\lambda} + \frac{\Delta - H_1 e^{-\lambda(H_1-\Delta)}}{1 - e^{-\lambda(H_1-\Delta)}}$$ Equation 30-57

$$H_1 = \frac{1.47\, S_f - u_{rt}}{r_d} + t_{cl} + \frac{L_h}{1.47\, S_f} \geq \Delta$$ Equation 30-58

$$\lambda = \frac{1}{\frac{1}{q_n} - \Delta}$$ Equation 30-59

where

u_m = minimum speed of the first through vehicle given that it is delayed (ft/s),

u_{rt} = right-turn speed = 20 (ft/s),

S_f = free-flow speed (mi/h),

$h_{|\Delta<h<H_1}$ = average headway of those headways between Δ and H_1 (s/veh),

Δ = headway of bunched vehicle stream = 1.5 (s/veh),

H_1 = maximum headway that the first through vehicle can have and still incur delay (s/veh),

r_d = deceleration rate = 6.7 (ft/s^2),

t_{cl} = clearance time of the right-turn vehicle = 0.6 (s),

L_h = average vehicle spacing in stationary queue (see Equation 30-15) (ft/veh),

λ = flow rate parameter (veh/s),

q_n = outside lane flow rate = v_n/3,600 (veh/s), and

v_n = flow rate for the outside lane (veh/h/ln).

The right-turn speed u_{rt} used in Equation 30-56 and Equation 30-58 is likely to be sensitive to access point design, including the approach profile, throat width, and curb radius. For level profiles and nominal throat widths, the speed can vary from 15 to 25 ft/s for radii varying from 20 to 60 ft, respectively. A default turn speed of 20 ft/s is recommended when information is not available to make a more accurate estimate.

The flow rate for the outside lane v_n is computed by using Steps 3, 4, 5, and 6 from the procedure described in the previous subsection, Delay due to Left Turns. However, the probability of a lane change P_{lc} is set equal to 1.0 when the calculations in Step 3 are made. In Steps 4 and 5, the variable I_t is set equal to 0.00001. The proportion of right-turning vehicles in the shared lane P_R is also computed at this point and used in a later step.

Step 2: Compute Delay to the First Through Vehicle

Equation 30-60
$$d_1 = \frac{(1.47\,S_f - u_m)^2}{2\,(1.47\,S_f)}\left(\frac{1}{r_d} + \frac{1}{r_a}\right)$$

where d_1 is the conditional delay to the first through vehicle (s/veh), and r_a is the acceleration rate = 3.5 (ft/s²).

Step 3: Compute Delay to the Second Through Vehicle

Equation 30-61
$$d_2 = d_1 - \left(h_{|\Delta<h<H_2} - \Delta\right)$$

with

Equation 30-62
$$h_{|\Delta<h<H_2} = \frac{1}{\lambda} + \frac{\Delta - H_2 e^{-\lambda(H_2-\Delta)}}{1 - e^{-\lambda(H_2-\Delta)}}$$

Equation 30-63
$$H_2 = d_1 + \Delta$$

where d_2 is the conditional delay to Vehicle 2 (s/veh).

Step 4: Compute Delay to the Third and Subsequent Through Vehicles

Equation 30-64
$$d_i = d_{i-1} - \left(h_{|\Delta<h<H_i} - \Delta\right)$$

with

Equation 30-65
$$h_{|\Delta<h<H_i} = \frac{1}{\lambda} + \frac{\Delta - H_i e^{-\lambda(H_i-\Delta)}}{1 - e^{-\lambda(H_i-\Delta)}}$$

Equation 30-66
$$H_i = d_{i-1} + \Delta$$

where d_i is the conditional delay to vehicle i (i = 3, 4, . . . ,) (s/veh). As shown by Equation 30-61 and Equation 30-64, the delay to each subsequent through vehicle is less than or equal to that of the preceding vehicle. In fact, the sequence of delays always converges to zero when the average flow rate in the outside lane is less than $1/\Delta$.

Step 4 should be repeated for the third and subsequent through vehicles until the delay computed for vehicle i is less than 0.1 s. In general, this criterion results in delay being computed for only the first two or three vehicles.

Step 5: Compute Through-Vehicle Delay per Right-Turn Maneuver

The through-vehicle delay for the first two vehicles is computed with Equation 30-67.

Equation 30-67
$$d_{t|r} = d_1\left(1 - e^{-\lambda(H_1-\Delta)}\right)(1 - P_R) + d_2\left(1 - e^{-\lambda(H_1-\Delta)}\right)\left(1 - e^{-\lambda(H_2-\Delta)}\right)(1 - P_R)^2$$

where $d_{t|r}$ is the through-vehicle delay per right-turn maneuver (s/veh). If three or more vehicles are delayed, an additional term needs to be added to Equation 30-67 for each subsequent vehicle. In this situation, Equation 30-68 can be used to compute the delay for any number of vehicles.

Equation 30-68
$$d_{t|r} = \sum_{i=1}^{\infty}\left[d_i \times \prod_{j=1}^{i}\left(1 - e^{-\lambda(H_j-\Delta)}\right) \times (1 - P_R)^i\right]$$

Step 6: Compute Through-Vehicle Delay due to Right Turns

The through-vehicle delay due to right turns $d_{ap,r}$ is computed with Equation 30-55.

5. PLANNING-LEVEL ANALYSIS APPLICATION

OVERVIEW OF THE APPLICATION

This section describes a simplified method for evaluating the operation of a coordinated street segment with signalized boundary intersections. The application addresses motorized vehicle operation. It is focused on the analysis of the through movement at the boundary intersections. This method can be used when minimal data are available for the analysis and only approximate results are desired.

REQUIRED DATA AND SOURCES

The overall data requirements are summarized in Exhibit 30-7. Some of the input requirements may be met by assumed values or default values. Other data items are site-specific and must be obtained in the field. The objective of using the planning-level analysis application is to minimize the need for the collection of detailed field data.

Exhibit 30-7
Required Input Data for the Planning-Level Analysis Application

Data Category	Location	Input Data Element
Traffic characteristics	Boundary intersection	Through-demand flow rate Through-saturation flow rate Volume-to-capacity ratio of the upstream movements
	Segment	Platoon ratio Midsegment flow rate Midsegment delay
Geometric design	Boundary intersection	Number of through lanes Upstream intersection width
	Segment	Number of through lanes Segment length Restrictive median length Nonrestrictive median length Proportion of segment with curb Number of access point approaches Proportion of segment with on-street parking
Signal control	Boundary intersection	Effective green-to-cycle-length ratio Cycle length
Other	Segment	Analysis period duration Speed limit

At a minimum, the analyst must provide traffic volumes and the approach-lane configuration for the subject intersection. Default values for several variables are specifically identified in the methodology and integrated into the method. These values have been selected to be generally representative of typical conditions. Additional default values are identified in Section 3 of Chapter 18, Urban Street Segments.

METHODOLOGY

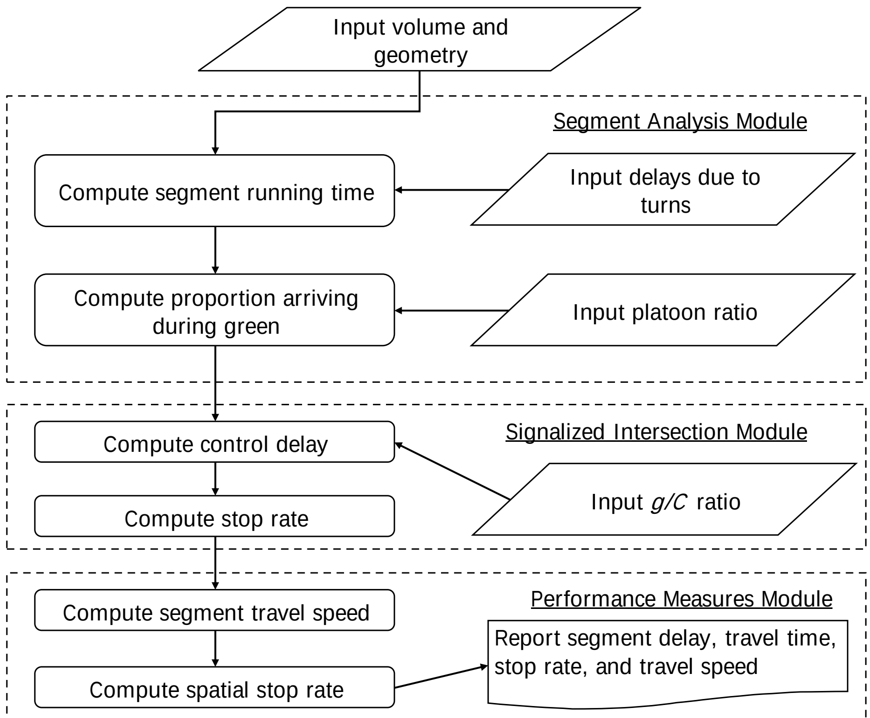
The methodology consists of five computational steps. These steps are

- Determine running time;
- Determine proportion arriving during green;
- Determine through control delay;

- Determine through stop rate; and
- Determine travel speed, spatial stop rate, and level of service (LOS).

Each step is executed in the sequence presented in the preceding list. This sequence is illustrated by the flowchart in Exhibit 30-8. The rectangles with rounded corners indicate the computational steps. The parallelograms indicate where input data are needed.

Exhibit 30-8
Planning-Level Analysis Application for Urban Street Segments



The computations associated with each step identified in Exhibit 30-8 are described in Section 3 of Chapter 18. These computations are conveniently illustrated here in a series of worksheets; each worksheet corresponds to one or two of the calculation steps.

The first of the computational worksheets is the Running Time worksheet. It is shown as Exhibit 30-9 (values shown apply to the Example Problem, as discussed in a subsequent section).

Exhibit 30-9
Planning-Level Analysis:
Running Time Worksheet

RUNNING TIME WORKSHEET			
General Information		**Site Information**	
Analyst	JME	Street	Texas Avenue
Agency or Company	ACME Engr.	Jurisdiction	
Date Performed	9/30/15	Analysis Year	2015
Analysis Time Period	5:30 p.m. to 5:45 p.m.	Analysis Level	planning
Base free-flow speed calibration factor (S_{calib}), mi/h: 0.0			

Input Data	Segment 1		Segment 2	
Direction of travel	EB/NB	WB/SB	EB/NB	WB/SB
Segment Data				
Number of through lanes for length of segment N_{th} (ln)	2	2		
Speed limit S_{pl} (mi/h)	35	35		
Midsegment volume v_m (veh/h)	1,150	1,150		
Total delay due to turns into access points Σd_{ap} (s/veh)	0.52	0.52		
Delay due to other midsegment sources d_{other} (s/veh)	0	0		
Length of segment L (ft)	1,800	1,800		
Width of upstream boundary intersection W_i (ft)	50	50		
Length of segment with restrictive median L_{rm} (ft)	0	0		
Length of segment with nonrestrictive median L_{nr} (ft)	0	0		
Start-up lost time l_1 (s)	2.0	2.0		
Access Data				
Proportion of segment with curb on right-hand side p_{curb}	0.70	0.70		
Number of access points on right-hand side N_{ap}	4	4		
Proportion of segment with on-street parking p_{pk}	0.00	0.00		
Running Time Computation				
Adjusted segment length L_{adj} (ft) $L_{adj} = L - W_i$	1,750	1,750		
Proportion of segment length with restrictive median p_{rm}, $p_{rm} = L_{rm} / L_{adj}$	0.0	0.0		
Speed constant S_0 (mi/h), $S_0 = 25.6 + 0.47\ S_{pl}$	42.1	42.1		
Adjustment for cross section f_{CS} (mi/h), $f_{CS} = 1.5\ p_{rm} - 0.47\ p_{curb} - 3.7\ p_{curb}\ p_{rm}$	-0.3	-0.3		
Access point density D_a (access points/mi), $D_a = 5{,}280\ (N_{ap,EB/NB} + N_{ap,WB/SB}) / L_{adj}$	24.1	24.1		
Adjustment for access points f_A (mi/h), $f_A = -0.078\ D_a / N_{th}$	-0.9	-0.9		
Adjustment for on-street parking f_{pk} (mi/h), $f_{pk} = -3\ p_{pk}$	0.0	0.0		
Base free-flow speed S_{fo} (mi/h), $S_{fo} = S_{calib} + S_0 + f_{CS} + f_A + f_{pk}$	40.8	40.8		
Segment length adjustment factor f_L, $f_L = 1.02 - 4.7\ (S_{fo} - 19.5)/\max(L, 400) \leq 1.0$	0.96	0.96		
Free-flow speed S_f (mi/h), $S_f = S_{fo}\ f_L \geq S_{pl}$	39.3	39.3		
Proximity adjustment factor f_v $\quad f_v = \dfrac{2}{1 + \left(1 - \dfrac{v_m}{52.8\ N_{th}\ S_f}\right)^{0.21}}$	1.03	1.03		
Running time t_R (s) $\quad t_R = \dfrac{6.0 - l_1}{0.0025\ L} + \dfrac{3{,}600\ L}{5{,}280\ S_f} f_v + \Sigma d_{ap} + d_{other}$	33.7	33.7		

Note: The first term in the running time equation is only applicable to segments with signal-controlled, STOP-controlled, or YIELD-controlled through movement at the boundary intersection.

The Running Time worksheet combines input data describing the segment geometric design, speed limit, volume, and access point frequency to estimate the base free-flow speed. This speed is then adjusted for segment length effects to obtain the expected free-flow speed. The free-flow speed is then used to estimate a free-flow travel time, which is adjusted for the proximity of other vehicles. Delay that is caused by turns into access points or other sources is added to the adjusted travel time. Default values for the delay due to turns at midsegment access points are listed in Exhibit 18-13 in Chapter 18. These defaults can be used when more accurate estimates of this delay are not available. The result of these adjustments is an estimate of the expected segment running time.

The second of the computational worksheets is the Proportion Arriving During Green worksheet. It is shown as Exhibit 30-10. This worksheet is designed for the analysis of the segment through-lane group. It documents the calculation of the proportion of vehicles that arrive during the green indication. Input data include the effective green-to-cycle-length ratio and platoon ratio.

Exhibit 30-10
Planning-Level Analysis: Proportion Arriving During Green Worksheet

PROPORTION ARRIVING DURING GREEN WORKSHEET	
General Information	
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.

Input Data				
	Segment 1		Segment 2	
Direction of travel	EB/NB	WB/SB	EB/NB	WB/SB
Signal Timing Data				
Effective green-to-cycle-length ratio g/C	0.47	0.47		
Traffic Data				
Platoon ratio R_p	1.43	0.67		
Proportion Arriving During Green Computation				
Proportion arriving during green P, $P = R_p\ (g/C)$	0.67	0.31		

The third computational worksheet is the Control Delay worksheet. It is shown as Exhibit 30-11. This worksheet is designed for the analysis of the segment through-lane group. Input variables include the analysis period duration, cycle length, effective green-to-cycle-length ratio, volume, saturation flow rate, and lanes. The proportion of arrivals during green is obtained from the previous worksheet.

The equation for computing the progression adjustment factor PF^* that is provided in Exhibit 30-11 is a simplified version of the exact equation (as provided in Section 3 of Chapter 19). The simplified equation, in combination with the supplemental adjustment factor f_{PA}, is sufficiently accurate for purposes of the planning-level analysis application.

The control delay is computed as the sum of two components. The first component to be computed is the uniform delay. The notation "min(1, X)" is shown in the equation used to compute this delay. It means that the value to be substituted for this text is the smaller of 1.0 and the volume-to-capacity ratio.

Exhibit 30-11
Planning-Level Analysis:
Control Delay Worksheet

CONTROL DELAY WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
Analysis period T (h): 0.25	Segment 1		Segment 2	
Direction of travel	EB/NB	WB/SB	EB/NB	WB/SB
Signal Timing Data				
Cycle length C (s)	100	100		
Effective green-to-cycle-length ratio g/C	0.47	0.47		
Traffic Data				
Through-lane group volume v_{th} (veh/h)	968	950		
Lane group saturation flow rate s (veh/h/ln)	1,800	1,800		
Proportion of arrivals during green P	0.67	0.31		
Volume-to-capacity ratio X_u of the upstream movements	0.57	0.57		
Geometric Design Data				
Number of through lanes N_{th} (ln)	2	2		
Delay Computation				
Capacity c (veh/h), $c = N_{th}\, s\, g/C$	1,692	1,692		
Volume-to-capacity ratio X, $X = v_{th}/c$	0.57	0.56		
Supplemental adjustment factor for platoons arriving during green f_{PA}, f_{PA} = 1.00 except as noted below: If $0.50 < R_p \le 0.85$, then $f_{PA} = 0.93$ If $1.15 < R_p \le 1.50$, then $f_{PA} = 1.15$	1.15	0.93		
Progression adjustment factor PF^*, $PF^* = f_{PA}\,(1 - P)/(1 - g/C)$	0.71	1.20		
Uniform delay d_1 (s/veh), $d_1 = \left(PF^*\right)\dfrac{0.5\,C\,(1-g/C)^2}{1-\left[\min(1,X)\,g/C\right]}$	13.6	23.0		
Upstream filtering adjustment factor I, $I = 1.0 - 0.91\,X_u^{2.68} \ge 0.090$	0.80	0.80		
Incremental delay d_2 (s/veh), $d_2 = 900\,T\left[(X-1)+\sqrt{(X-1)^2+\dfrac{4\,I\,X}{c\,T}}\right]$	1.13	1.08		
Control delay d (s/veh), $d = d_1 + d_2$	14.7	24.1		

The second delay component is the incremental delay, which is based on the upstream filtering adjustment factor. This factor requires the variable X_u, which can be estimated as the volume-to-capacity ratio of the segment through-lane group at the upstream signalized intersection. Additional detail on the calculation of this ratio is provided in Section 3 of Chapter 19, Signalized Intersections.

The fourth computational worksheet is the Stop Rate worksheet. It is shown as Exhibit 30-12. This worksheet is designed for the analysis of the segment through-lane group. The input variables are the same as those needed for the Control Delay worksheet with the addition of speed limit. The average speed during the analysis period is estimated by using the equation provided. If the average speed is known, it should be substituted for the estimated value.

Exhibit 30-12
Planning-Level Analysis: Stop Rate Worksheet

STOP RATE WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
Analysis period T (h): 0.25	Segment 1		Segment 2	
Direction of travel	EB/NB	WB/SB	EB/NB	WB/SB
Signal Timing Data				
Cycle length C (s)	100	100		
Effective green-to-cycle-length ratio g/C	0.47	0.47		
Traffic Data				
Through-lane group volume v_{th} (veh/h)	968	950		
Lane group saturation flow rate s (veh/h/ln)	1,800	1,800		
Proportion of arrivals during green P	0.67	0.31		
Speed limit S_{pl} (mi/h)	35	35		
Incremental delay d_2 (s/veh)	1.13	1.08		
Geometric Design Data				
Number of through lanes N_{th} (ln)	2	2		
Stop Rate Computation				
Effective green time g (s), $g = C\,(g/C)$	47	47		
Effective red time r (s), $r = C - g$	53	53		
Capacity c (veh/h), $c = N_{th}\, s\, g/C$	1,692	1,692		
Volume-to-capacity ratio X, $X = v_{th}/c$	0.57	0.56		
Average speed S_a (mi/h), $S_a = 0.90\,(25.6 + 0.47\,S_{pl})$	37.8	37.8		
Threshold acceleration–deceleration delay (s), $(1 - P)\,g\,X$	8.8	18.1		
Acceleration–deceleration delay d_a (s), $d_a = 0.393\,(S_a - 5.0)^2 / S_a$	11.2	11.2		
Deterministic stop rate h_1 (stops/veh), $h_1 = \frac{1 - P\,(1 + d_a / g)}{1 - P\,X}$ if $d_a \le (1-P)\,g\,X$; $h_1 = \frac{(1-P)(r - d_a)}{r - (1-P)\,g\,X}$ if $d_a > (1-P)\,g\,X$	0.31	0.74		
Second-term back-of-queue size Q_2 (veh/ln), $Q_2 = c\,d_2 / (3{,}600\,N_{th})$	0.26	0.25		
Full stop rate h (stops/veh), $h = h_1 + 3{,}600\,N_{th}\,Q_2 / (v_{th}\,C)$	0.33	0.76		

The stop rate is computed as the sum of two components. The first component to be computed is the deterministic stop rate. Two equations are available for this computation. The correct equation to use is based on a check of the acceleration–deceleration delay relative to the computed threshold value.

The second stop rate component is based on the second-term back-of-queue size. This queue represents the average number of vehicles that are unserved at the end of the green interval. It is based on the incremental delay computed for the Control Delay worksheet.

The fifth computational worksheet is the Travel Speed and Spatial Stop Rate worksheet. It is shown as Exhibit 30-13. This worksheet is designed for the analysis of the segment through-lane group. The input values include segment length and the full stop rate associated with other midsegment events (e.g., turns at access points). The other input data listed represent computed values and are obtained from the previous worksheets.

Exhibit 30-13
Planning-Level Analysis: Travel Speed and Spatial Stop Rate Worksheet

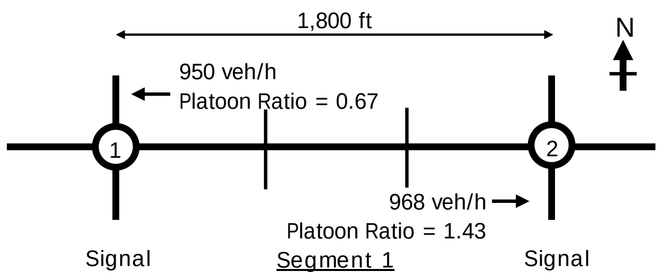
TRAVEL SPEED AND SPATIAL STOP RATE WORKSHEET				
General Information				
Project Description	Texas Avenue, 5:30 p.m. to 5:45 p.m.			
Input Data				
	Segment 1		Segment 2	
Direction of travel	EB/NB	WB/SB	EB/NB	WB/SB
Length of segment L (ft)	1,800	1,800		
Base free-flow speed S_{fo} (mi/h)	40.8	40.8		
Running time t_R (s)	33.7	33.7		
Control delay d (s/veh)	14.7	24.1		
Full stop rate h (stops/veh)	0.33	0.76		
Full stop rate due to other midsegment sources h_{other} (stops/veh)	0	0		
Travel Speed Computation				
Travel time T_T (s), $T_T = t_R + d$	48.4	57.7		
Travel speed $S_{T,seg}$ (mi/h), $S_{T,seg} = \frac{3{,}600\,L}{5{,}280\,T_T}$	25.4	21.3		
Spatial Stop Rate Computation				
Total stop rate h_T (stops/veh), $h_T = h + h_{other}$	0.33	0.76		
Spatial stop rate H_{seg} (stops/mi), $H_{seg} = \frac{5{,}280\,h_T}{L}$	0.96	2.23		
Level-of-Service Computation				
Volume-to-capacity ratio X, $X = v_{th}/c$	0.57	0.56		
Travel speed thresholds for base free-flow speed (S_{fo}) by interpolation of values in Exhibit 18-1 (mi/h)	A: >32.6 B: >27.3 C: >20.4 D: >16.3 E: >12.2	A: >32.6 B: >27.3 C: >20.4 D: >16.3 E: >12.2		
Level of service	C	C		

EXAMPLE PROBLEM

The Urban Street Segment

The total length of an undivided urban street segment is 1,800 ft. It is shown in Exhibit 30-14. Both of the boundary intersections are signalized. The street has a four-lane cross section with two lanes in each direction. There are left-turn bays on the subject segment at each signalized intersection.

Exhibit 30-14
Planning-Level Analysis: Example Problem



The segment has two access point intersections. Each intersection has two STOP-controlled side-street approaches, and each approach has sufficient traffic volume during the analysis period to be considered active. The segment also has two driveways on each side of the street; however, their turn movement volumes are too low for them to be considered active.

The Question

What are the travel speed, spatial stop rate, and LOS during the analysis hour for through-vehicle traffic in both directions of travel along the segment?

The Facts

Some details of the segment are shown in Exhibit 30-14. Both boundary intersections are signalized. The following additional information is known about the street segment:

Through saturation flow rate: 1,800 veh/h/ln

Midsegment volume: 1,150 veh/h

Midsegment delay: 0.52 s/veh

Number of through lanes at boundary intersection: 2

Upstream intersection width: 50 ft

Number of through lanes on segment: 2

Proportion of street with curb: 0.70

Proportion of street with on-street parking: 0.0

g/C ratio: 0.47

Cycle length: 100 s

Analysis period: 0.25 h

Speed limit: 35 mi/h

Percent left turns at active access points: 6%

Percent right turns at active access points: 8%

Selected Calculations

1. Compute total delay due to turns into access points	Midsegment lanes = 2 lanes Midsegment lane volume = 575 veh/h/ln Interpolate in Exhibit 18-13 to obtain 0.37 s/veh/pt through-vehicle delay. Number of active access points = 2 Percent turns = 7% [= (6 + 8)/2] Total delay per access pt. = 7/10 × 0.37 = 0.26 s/veh/pt Total delay per segment = 2 × 0.26 = 0.52 s/veh
2. Compute upstream filtering factor	No information was available about the volume-to-capacity ratio for the upstream movements, so this ratio was estimated to equal the volume-to-capacity ratio for the subject movement.

Results

The calculations are shown in Exhibit 30-9 to Exhibit 30-13. The travel speed for the eastbound direction is 25.4 mi/h. The travel speed for the westbound direction is 21.3 mi/h. The eastbound and westbound spatial stop rates are 0.96 and 2.23 stops/mi, respectively.

The base free-flow speed is 40.8 mi/h. By interpolating this value between those in Exhibit 18-1, the threshold travel speeds for LOS A, B, C, D, and E are >32.6, >27.3, >20.4, >16.3, and >12.2 mi/h, respectively. Thus, the travel speed for the eastbound direction of 26.3 mi/h corresponds to LOS C. The westbound LOS is similarly determined to be C.

6. FIELD MEASUREMENT TECHNIQUES

This section describes two techniques for estimating key vehicular traffic characteristics by using field data. The first technique is used to estimate free-flow speed. The second technique is used to estimate average travel speed.

The field measurements for both techniques should occur during a time period that is representative of the analysis period. This approach recognizes a possible difference in driver speed choice during different times of day (and, possibly, days of week and months of year).

FREE-FLOW SPEED

The following steps can be used to determine the free-flow speed for vehicular traffic on an urban street segment. The definition of "urban street segment" is provided in Section 2 of Chapter 18.

The speed measured with the technique described in this section describes the free-flow speed for the subject segment. It is not necessarily an accurate measurement of the free-flow speed on an adjacent segment because of possible differences in geometry, access point spacing, or speed limit.

Some urban streets have characteristics that can influence free-flow speed but that are not considered in the predictive procedure. If free-flow speed is measured for these segments, the results should be qualified to acknowledge the possible influence of these characteristics on the measured speed. These characteristics include a change in the posted speed limit along the segment, the display of an advisory speed sign that has an advisory speed lower than the speed limit, a change in the number of through lanes along the segment, significant grade, or a midsegment capacity constraint (e.g., narrow bridge).

Step 1. Conduct a spot-speed study at a midsegment location during low-volume conditions. Record the speed of 100 or more free-flowing passenger cars. A car is free-flowing when it has a headway of 8 s or more to the vehicle ahead and 5 s or more to the vehicle behind in the same traffic lane. In addition, a free-flow vehicle is not influenced (i.e., slowed) by the following factors: (*a*) vehicles turning onto (or off of) the subject segment at the boundary intersection or at a midsegment access point, (*b*) traffic control devices at the boundary intersections, or (*c*) traffic control devices deployed along the segment.

In view of the aforementioned definition of "free-flow vehicle," vehicles turning into (or out of) an access point should not be included in the database. Vehicles that are accelerating or decelerating as a result of driver response to a traffic control signal should not be included in the database. Vehicles should not be included if they are influenced by signs that require a lower speed limit during school hours or signs that identify a railroad crossing.

Step 2. Compute the average of the spot speeds S_{spot} and their standard deviation σ_{spot}.

Step 3. Compute the segment free-flow speed S_f as a space mean speed by using Equation 30-69.

Equation 30-69

$$S_f = S_{\text{spot}} - \frac{\sigma_{\text{spot}}^2}{S_{\text{spot}}}$$

where

S_f = free-flow speed (mi/h),

S_{spot} = average spot speed (mi/h), and

σ_{spot} = standard deviation of spot speeds (mi/h).

Step 4. If the base free-flow speed S_{fo} is also desired, it can be computed by using Equation 30-70.

Equation 30-70

$$S_{fo} = \frac{S_f}{f_L}$$

with

Equation 30-71

$$f_L = 1.02 - 4.7\frac{S_f - 19.5}{\max(L_s, 400)} \leq 1.0$$

where

S_{fo} = base free-flow speed (mi/h),

S_f = free-flow speed (mi/h),

L_s = distance between adjacent signalized intersections (ft), and

f_L = signal spacing adjustment factor.

Equation 30-71 was originally derived with the intent of using the base free-flow speed S_{fo} in the numerator of the second term. However, use of the free-flow speed S_f in its place is sufficient for this application.

Equation 30-71 was derived by using signalized boundary intersections. For more general applications, the definition of distance L_s is broadened so that it equals the distance between the two intersections that (*a*) bracket the subject segment and (*b*) each have a type of control that can impose on the subject through movement a legal requirement to stop or yield.

AVERAGE TRAVEL SPEED

The following steps can be used to determine the average travel speed for vehicular traffic on an urban street segment.

Step 1. Identify the time of the day (e.g., morning peak, evening peak, off-peak) during which the study will be conducted. Identify the segments to be evaluated.

Step 2. Conduct the test car travel time study for the identified segments during the identified study period. The following factors should be considered before or during the field study:

- The number of travel time runs will depend on the range of speeds found on the street. Six to 12 runs for each traffic volume condition are typically adequate. The analyst should determine the minimum number of runs on the basis of guidance provided elsewhere (*8*).

- The objective of the data collection is to obtain the information identified in the Travel Time Field Worksheet (i.e., vehicle location and arrival and departure times at each boundary intersection). This worksheet is shown in Exhibit 30-15. In general, each row of this worksheet represents the data for one direction of travel on one segment. If the street serves traffic in two travel directions, separate worksheets are typically used to record the data for each direction of travel.
- The equipment used to record the data may include a Global Positioning System–equipped laptop computer or simply a pair of stopwatches. If available, an instrumented test car should be used to reduce labor requirements and to facilitate recording and analysis.
- During the test run, the average-car technique is typically used and requires that the test car travel at the average speed of the traffic stream, as judged by its driver (*8*).
- The cumulative travel time is recorded as the vehicle passes the center of each boundary intersection. Whenever the test car stops or slows (i.e., 5 mi/h or less), the observer uses a second stopwatch to measure the duration of time the vehicle is stopped or slowed. This duration (and the cause of the delay) is recorded on the worksheet on the same row that is associated with the next boundary intersection to be reached. The rows are intentionally tall so that a midsegment delay and the signal delay can both be recorded in the same cell.
- Test car runs should begin at different time points in the signal cycle to avoid having all runs start from a "first in platoon" position.
- Some midsegment speedometer readings should also be recorded to check on unimpeded travel speeds and to see how they relate to the estimated free-flow speed.

Step 3. The cumulative travel time observations between adjacent boundary intersections are subtracted to obtain the travel time for the corresponding segment. This travel time can be averaged for all test runs to obtain an average segment travel time. The average is then divided into the segment length to obtain an estimate of the average travel speed. This speed should be computed for each direction of travel for the segment.

The data should be summarized to provide the following statistics for each segment travel direction: average travel speed, average delay time for the boundary intersection, and average delay time for other sources (pedestrian, parking maneuver, etc.).

The average segment travel time for each of several consecutive segments in a common direction of travel can be added to obtain the total travel time for the facility. This total travel time can then be divided into the facility length (i.e., the total length of all segments) to obtain the average travel speed for the facility. This calculation should be repeated to obtain the average travel speed for the other direction of travel.

Exhibit 30-15
Travel Time Field Worksheet

TRAVEL TIME FIELD WORKSHEET

General Information		Site Information	
Analyst	________	Street	________
Agency or Company	________	Jurisdiction	________
Date Performed	________	Analysis Year	________
Analysis Time Period		Direction of Travel	

Field Data

Location (typically a boundary intersection)	Run Number: __________________			Run Number: ________________		
	Cumulative Travel Time at Location (s)	Delays due to Slow or Stop		Cumulative Travel Time at Location (s)	Delays due to Slow or Stop	
		Cause [a]	Delay Time (s)		Cause [a]	Delay Time (s)

Notes: [a] Cause of delay: Ts = signal; Lt = left turn; Pd = pedestrian; Pk = parking; Ss = STOP sign; Ys = YIELD sign.

7. COMPUTATIONAL ENGINE DOCUMENTATION

This section uses a series of flowcharts and linkage lists to document the logic flow for the computational engine.

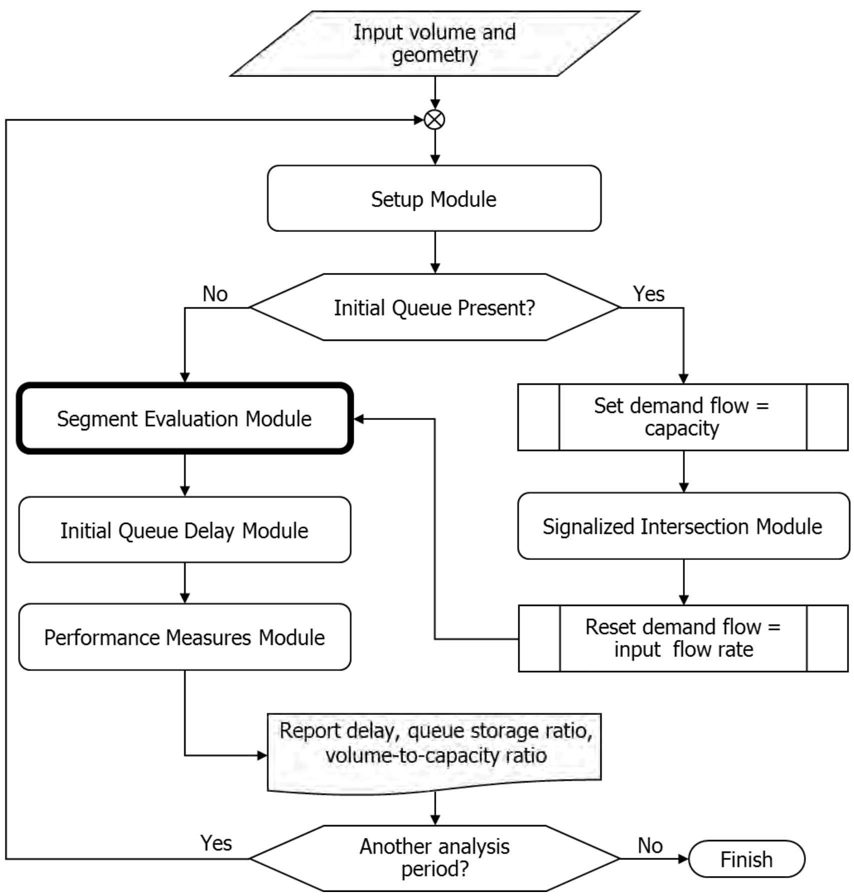
FLOWCHARTS

The methodology flowchart is shown in Exhibit 30-16. The methodology consists of five main modules:

- Setup Module,
- Segment Evaluation Module,
- Segment Analysis Module,
- Delay due to Turns Module, and
- Performance Measures Module.

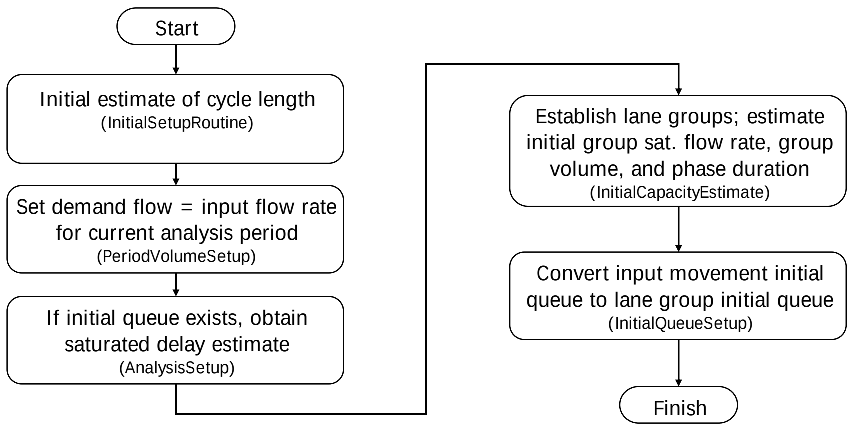
This subsection provides a separate flowchart for each of these modules.

Exhibit 30-16
Methodology Flowchart



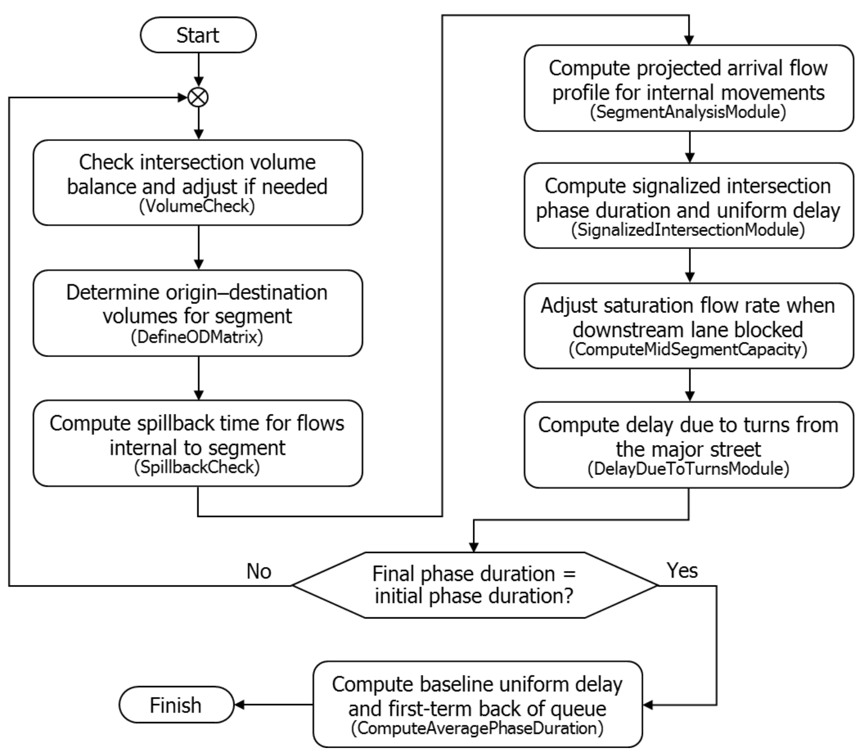
The Setup Module is shown in Exhibit 30-17. This module consists of five main routines, as shown in the large rectangles of the exhibit. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines and the Initial Queue Delay Module are described in Chapter 31, Signalized Intersections: Supplemental.

Exhibit 30-17
Setup Module



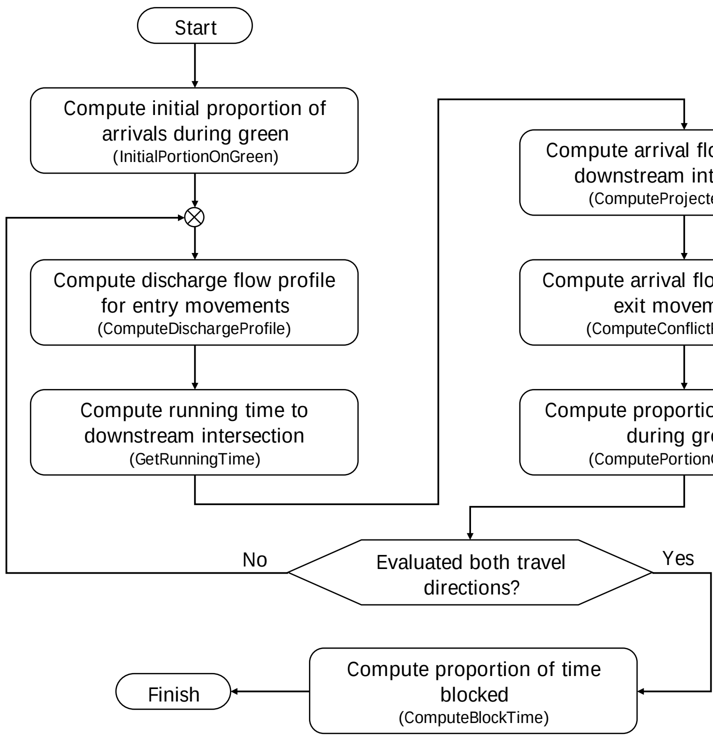
The Segment Evaluation Module is shown in Exhibit 30-18. This module consists of eight main routines. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. The Segment Analysis Module and the Delay due to Turns Module are outlined in the next two exhibits. The Signalized Intersection Module and the Compute Average Phase Duration routine are described in Chapter 31. The Volume Check, Define Origin–Destination Matrix, Spillback Check, and Midsegment Capacity routines are described further in the next subsection.

Exhibit 30-18
Segment Evaluation Module



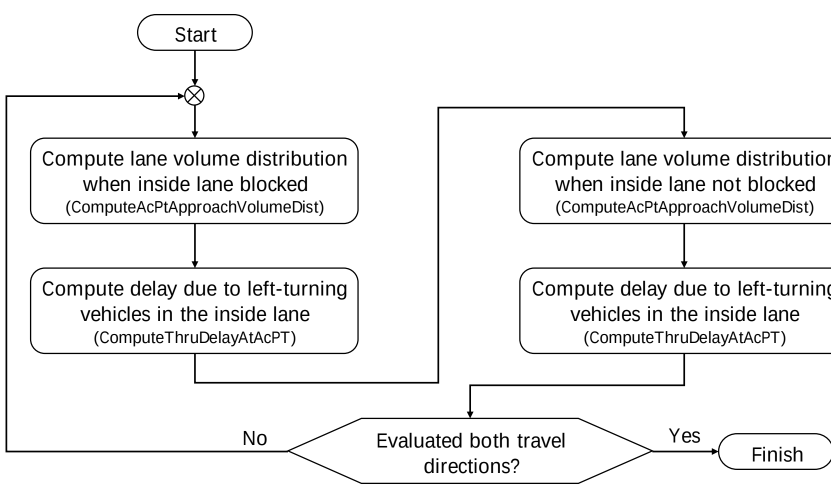
The Segment Analysis Module is shown in Exhibit 30-19. This module consists of seven main routines, six of which are implemented for both segment travel directions. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines are described further in the next subsection.

Exhibit 30-19
Segment Analysis Module



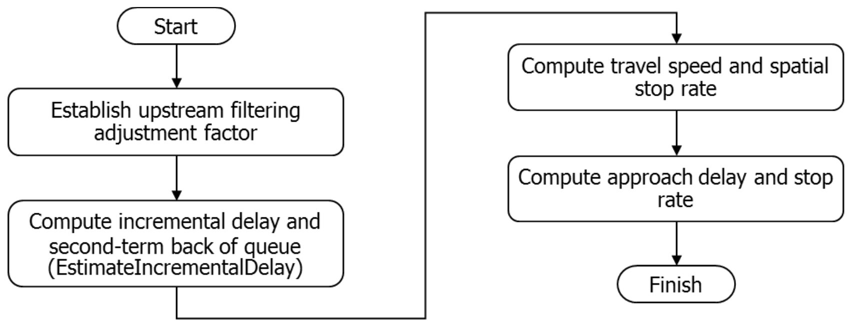
The Delay due to Turns Module is shown in Exhibit 30-20. This module consists of two main routines, each of which is implemented for both segment travel directions. The main function of each routine, as well as the name given to it in the computational engine, is also shown in the exhibit. These routines are described further in the next subsection.

Exhibit 30-20
Delay due to Turns Module



The Performance Measures Module is shown in Exhibit 30-21. This module consists of four routines. The main function of each routine is also shown in the exhibit. One of the routines (i.e., EstimateIncrementalDelay) is complicated enough to justify its development as a separate entity in the computational engine. This routine is described in Chapter 31, Signalized Intersections: Supplemental.

Exhibit 30-21
Performance Measures Module



LINKAGE LISTS

This subsection uses linkage lists to describe the main routines that make up the computational engine. Each list is provided in a table that identifies the routine and the various subroutines that it references. Conditions for which the subroutine is used are also provided.

The lists are organized by module, as described in the previous subsection. A total of three tables are provided to address the following three modules:

- Segment Evaluation Module,
- Segment Analysis Module, and
- Delay due to Turns Module.

The linkage list for the Segment Evaluation Module is provided in Exhibit 30-22. The main routines are listed in Column 1 and were previously identified in Exhibit 30-18.

The linkage list for the Segment Analysis Module is provided in Exhibit 30-23. The main routines are listed in Column 1 and were previously identified in Exhibit 30-19.

Finally, the linkage list for the Delay due to Turns Module is provided in Exhibit 30-24. The main routines are listed in Column 1 and were previously identified in Exhibit 30-20.

Exhibit 30-22
Segment Evaluation Module Routines

Routine	Subroutine	Conditions for Use
VolumeCheck	Ensure that discharge volume for each entry movement does not exceed its capacity.	Apply for both segment travel directions.
DefineODMatrix	ComputeODs (compute origin–destination volume for movements that enter and exit segment)	Apply to all intersections on segment and for both segment travel directions.
SpillbackCheck	ComputeSpillbackTime (compute spillback time for each exit movement at the downstream boundary intersection)	Apply for both segment travel directions.
SegmentAnalysisModule	See Exhibit 30-23.	
SignalizedIntersectionModule	See Chapter 31.	
ComputeMidSegmentCapacity	Compute midsegment capacity when restricted and reduce saturation flow rate of upstream movements so upstream discharge is less than or equal to the midsegment capacity.	Apply to each upstream signalized intersection traffic movement that enters segment.
DelayDueToTurnsModule	See Exhibit 30-24.	
ComputeAveragePhaseDuration	See Chapter 31.	

Exhibit 30-23
Segment Analysis Module Routines

Routine	Subroutine	Conditions for Use
InitialPortionOnGreen	Compute proportion of arrivals during green (*P*) based on current signal timing.	None
ComputeDischargeProfile	Compute discharge flow rate for each 1-s interval of signal cycle at upstream boundary intersection.	Apply to each upstream boundary intersection movement that enters segment.
GetRunningTime	Compute running time on length of street between upstream boundary intersection and subject downstream intersection.	Apply to all intersections on the segment and for both segment travel directions.
ComputeProjectedProfile	Compute arrival flow profile reflecting dispersion of platoons formed at upstream boundary intersection.	Apply to each upstream boundary intersection movement that enters segment.
ComputeConflictFlowRate	Use arrival flow profile and origin–destination matrix to compute arrival flow rate for movements at subject intersection.	Apply to all intersections on the segment and for both segment travel directions.
	Compute conflicting flow rate at access point intersections on basis of the projected arrivals at each intersection.	Apply to all access point intersections and for both segment travel directions.
ComputePortionOnGreen	For each exit movement, compute count of vehicles arriving at downstream boundary intersection during green.	Apply to each downstream boundary intersection.
ComputeBlockTime	Use computed conflicting flow rates at each access point intersection to compute the proportion of time blocked for each nonpriority movement.	Apply to all access point intersections and for both travel segment travel directions.

Exhibit 30-24
Delay due to Turns Module Routines

Routine	Subroutine	Conditions for Use
ComputeAcPtApproach-VolumeDist	Compute the volume for each lane on the approach to the access point intersection when blocked by a left-turning vehicle.	Apply lane volume routine for case in which inside lane is blocked by a turning vehicle. Apply to all access point intersections and for both segment travel directions.
	Compute the volume for each lane on the approach to the access point intersection when *not* blocked by a left-turning vehicle.	Apply lane volume routine for case in which inside lane is *not* blocked by a turning vehicle. Apply to all access point intersections and for both segment travel directions.
ComputeThruDelayAtAcPT	Compute the probability of left-turn bay overflow at access point intersection.	If segment is undivided, the probability of bay overflow is 1.0.
	Compute the delay to through movements due to a left turn at an access point. Based on lane volume estimate for case in which inside lane is blocked by a turning vehicle.	Apply to all access point intersections and for both segment travel directions.
	Compute the delay to through movements due to a right turn at an access point. Based on lane volume estimate for case in which inside lane is *not* blocked by a turning vehicle.	Apply to all access point intersections and for both segment travel directions.

8. EXAMPLE PROBLEMS

This section describes the application of each of the motorized vehicle, pedestrian, bicycle, and transit methodologies through the use of example problems. Exhibit 30-25 provides an overview of these problems. The focus of the examples is on an operational analysis. A planning and preliminary engineering analysis is identical to the operational analysis in terms of the calculations, except that default values are used when field-measured values are not available.

Exhibit 30-25
Example Problems

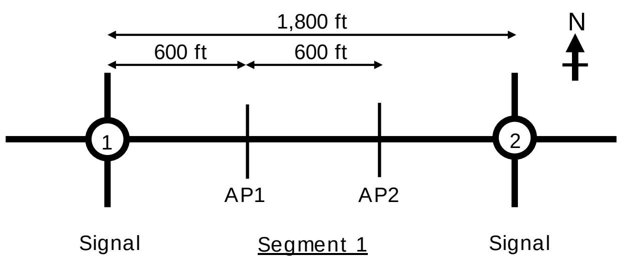
Problem Number	Description	Analysis Type
1	Motorized Vehicle LOS	Operational
2	Pedestrian LOS	Operational
3	Bicycle LOS	Operational
4	Transit LOS	Operational

EXAMPLE PROBLEM 1: MOTORIZED VEHICLE LOS

The Urban Street Segment

The total length of an undivided urban street segment is 1,800 ft. The segment is shown in Exhibit 30-26. Both of the boundary intersections are signalized. The street has a four-lane cross section with two lanes in each direction. There are left-turn bays on the subject segment at each signalized intersection.

Exhibit 30-26
Example Problem 1: Urban Street Segment Schematic



The segment has two active access point intersections, shown in the exhibit as AP1 and AP2. Each intersection has two STOP-controlled side-street approaches. The segment has some additional driveways on each side of the street; however, their turn movement volumes are too low during the analysis period for them to be considered active. The few vehicles that do turn at these locations during the analysis period have been added to the corresponding volumes at the two active access point intersections.

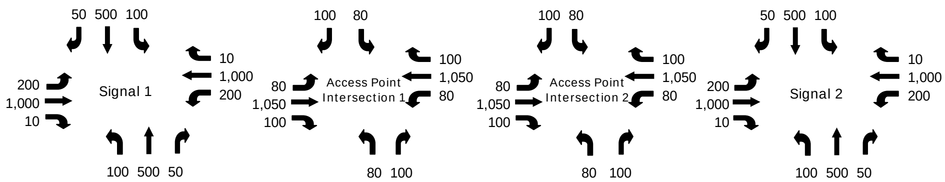
The Question

What are the travel speed, spatial stop rate, and LOS during the analysis period for the segment through movement in both directions of travel?

The Facts

The segment's traffic counts are listed in Exhibit 30-27. The counts were taken during the 15-min analysis period of interest. However, they have been converted to hourly flow rates. Note that the volumes leaving the signalized intersections do not add up to the volume arriving at the downstream access point intersection.

Exhibit 30-27
Example Problem 1: Intersection Turn Movement Counts



The signalization conditions are shown in Exhibit 30-28. The conditions shown are identified as belonging to Signalized Intersection 1; however, they are the same for Signalized Intersection 2. The signals operate with coordinated–actuated control. The left-turn movements on the northbound and southbound approaches operate under protected–permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the major street operate as protected-only in a lead–lead sequence.

Exhibit 30-28
Example Problem 1: Signal Conditions for Intersection 1

Controller Data Worksheet

General Information					
Cross street:	First Avenue		Analysis period:	7:15 am to 7:30 am	
Phase Sequence					
Phases 1 and 2			***Phases 3 and 8***		
		1. WB left (1) with WB thru (6)			1. NB left (3) with NB thru (8)
Enter choice	2	2. WB left (1) before EB thru (2)	Enter choice	2	2. NB left (3) before SB thru (4)
		3. EB thru (2) before WB left (1)			3. SB thru (4) before NB left (3)
Phases 5 and 6			***Phases 4 and 7***		
		1. EB left (5) with EB thru (2)			1. SB left (7) with SB thru (4)
Enter choice	2	2. EB left (5) before WB thru (6)	Enter choice	2	2. SB left (7) before NB thru (8)
		3. WB thru (6) before EB left (5)			3. NB thru (8) before SB left (7)
Left-Turn Mode					
Phase 1 or 2			***Phase 3 or 8***		
Enter choice	3	2. WB left (1) prot-perm	Enter choice	2	2. NB left (3) prot-perm
		3. WB left (1) protected			3. NB left (3) protected
Phase 5 or 6			***Phase 4 or 7***		
Enter choice	3	2. EB left (5) prot-perm	Enter choice	2	2. SB left (7) prot-perm
		3. EB left (5) protected			3. SB left (7) protected

Phase Settings								
Approach	**Eastbound**		**Westbound**		**Northbound**		**Southbound**	
Phase number	5	2	1	6	3	8	7	4
Movement	L	T+R	L	T+R	L	T+R	L	T+R
Lead/lag left-turn phase	Lead	--	Lead	--	Lead	--	Lead	--
Left-turn mode	Prot.	--	Prot.	--	Pr/Pm	--	Pr/Pm	--
Passage time, s	2.0		2.0		2.0	2.0	2.0	2.0
Phase split, s	20	35	20	35	20	25	20	25
Minimum green, s	5		5		5	5	5	5
Yellow change, s	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Red clearance, s	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Walk+ ped. clear, s		0		0		0		0
Recall?	No		No		No	No	No	No
Dual entry ?	No	Yes	No	Yes	No	Yes	No	Yes
Simultaneous gap-out?	Yes				Yes			
Dallas left-turn phasing?					No			
Coordination settings	Offset, s:	0	Offset Ref.:	End of Green		Force Mode:		Fixed
	Cycle, s:	100				Reference phase:		2

Exhibit 30-28 indicates that the passage time for each actuated phase is 2.0 s. The minimum green setting for each actuated phase is 5 s. The offset to Phase 2 (the reference phase) end-of-green interval is 0.0 s. A fixed-force mode is used to ensure that good coordination is maintained. The cycle length is 100 s.

Geometric conditions and traffic characteristics for Signalized Intersection 1 are shown in Exhibit 30-29. They are the same for Signalized Intersection 2. The movement numbers follow the numbering convention shown in Exhibit 19-1 of Chapter 19.

Exhibit 30-29
Example Problem 1: Geometric Conditions and Traffic Characteristics for Signalized Intersection 1

Intersection Data Worksheet												
Approach	Eastbound			Westbound			Northbound			Southbound		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Intersection Geometry												
Number of lanes	1	2	1	1	2	1	1	2	0	1	2	0
Lane assignment	L	T	R	L	T	R	L	TR	n.a.	L	TR	n.a.
Average lane width, ft	12.0	12.0	12.0	12.0	12.0	12.0	12.0	12.0		12.0	12.0	
Number of receiving lanes		2			2			2			2	
Turn bay or segment length, ft	200	999	200	200	1800	200	200	999		200	999	
Traffic Characteristics												
Volume, veh/h	200	1000	10	200	1000	10	100	500	50	100	500	50
Right-turn-on-red volume, veh/h			0			0			0			0
Percent heavy vehicles, %	0	0	0	0	0	0	0	0	0	0	0	0
Lane utilization adjustment factor		1.000			1.000							
Peak hour factor 1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Start-up lost time, s	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0		2.0	2.0	
Extension of eff. green time, s	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0		2.0	2.0	
Platoon ratio	1.000	1.333	1.000				1.000	1.000		1.000	1.000	
Upstream filtering factor	1.00	1.00	1.00				1.00	1.00		1.00	1.00	
Pedestrian volume, p/h		0			0			0			0	
Bicycle volume, bicycles/h		0			0			0			0	
Opposing right-turn lane influence	Yes			Yes								
Initial queue, veh	0	0	0	0	0	0	0	0		0	0	
Speed limit, mi/h	35	35	35	35	35	35	35	35		35	35	
Unsignalized movement volume, veh/h	0	0	0	0	0	0	0	0	0	0	0	0
Unsignalized movement delay, s/veh	0	0	0	0	0	0	0	0	0	0	0	0
Unsignalized mvmt. stop rate, stops/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Approach Data	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*
Parking present?	No		No	No		No	No		No	No		No
Parking maneuvers, maneuvers/h												
Bus stopping rate, buses/h			0			0			0			0
Approach grade, %	0	0	0	0	0	0	0	0	0	0	0	0
Detection Data												
Stop line detector presence	Presence			Presence			Presence	Presence		Presence	Presence	
Stop line detector length, ft	40			40			40	40		40	40	

All signalized intersection approaches have a 200-ft left-turn bay and two through lanes. The east–west approaches have a 200-ft right-turn lane. The north–south approaches have a shared through and right-turn lane. Many of the geometric and traffic characteristics shown in the exhibit are needed to compute the saturation flow rate with the procedure described in Section 3 of Chapter 19.

The platoon ratio is entered for all movements associated with an external approach to the segment. The eastbound through movement at Signalized Intersection 1 is known to be coordinated with the upstream intersection so that favorable progression occurs, as described by a platoon ratio of 1.333. The westbound through movement at Signalized Intersection 2 is also coordinated with its upstream intersection, and arrivals are described by a platoon ratio of 1.33. Arrivals to all other movements are characterized as “random” and are described with a platoon ratio of 1.00. The movements for the westbound approach at Signalized Intersection 1 (and eastbound approach at Signalized Intersection 2) are internal movements, so a platoon ratio (and upstream filtering factor) is not entered for them. More accurate values are computed during subsequent iterations by using a procedure provided in the methodology.

The speed limit on the segment and on the cross-street approaches is 35 mi/h. With a couple of exceptions, detection is located just upstream of the stop line in each traffic lane at the two signalized intersections. A 40-ft detection zone is used in each instance. The exceptions are the traffic lanes serving the major-street through movement at each intersection. There is no detection for these movements because they are not actuated.

The geometric conditions that describe the segment are shown in Exhibit 30-30. These data are used to compute the free-flow speed for the segment.

Exhibit 30-30
Example Problem 1: Segment Data

Segment Data Worksheet		
Input Data		
	EB	**WB**
Basic Segment Data		
Number of through lanes that extend the length of the segment:	2	2
Speed limit, mph	35	35
Segment Length Data		
Length of segment (measured stopline to stopline), ft	1800	1800
Width of upstream signalized intersection, ft	50	50
Adjusted segment length, ft	1750	1750
Length of segment with a restrictive median (e.g, raised-curb), ft	0	0
Length of segment with a non-restrictive median (e.g, two-way left-turn lane), ft	0	0
Length of segment with no median, ft	1750	1750
Percentage of segment length with restrictive median, %	0	0
Access Data		
Percentage of street with curb on right-hand side (in direction of travel), %	70	70
Number of access points on right-hand side of street (in direction of travel)	4	4
Percentage of street with on-street parking on right-hand side (in direction of travel),%	0	0
Other Delay Data		
Mid-segment delay, s/veh	0	0

The traffic and lane assignment data for the two access point intersections are shown in Exhibit 30-31. The movement numbers follow the numbering convention shown in Exhibit 20-1 of Chapter 20, Two-Way STOP-Controlled Intersections. There are no turn bays on the segment at the two access point intersections.

Exhibit 30-31
Example Problem 1: Access Point Data

Access Point Input Data													
Access	Approach	Eastbound			Westbound			Northbound			Southbound		
Point	Movement	L	T	R	L	T	R	L	T	R	L	T	R
Location,ft	Movement number	1	2	3	4	5	6	7	8	9	10	11	12
600	Volume, veh/h	80	1,050	100	80	1,050	100	80	0	100	80	0	100
West end	Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1200	Volume, veh/h	80	1,050	100	80	1,050	100	80	0	100	80	0	100
	Lanes	0	2	0	0	2	0	1	0	1	1	0	1

Outline of Solution

Movement-Based Data

Exhibit 30-32 provides a summary of the analysis of the individual traffic movements at Signalized Intersection 1.

Exhibit 30-32
Example Problem 1: Movement-Based Output Data

INTERSECTION 1	EB	EB	EB	WB	WB	WB	NB	NB	NB	SB	SB	SB
	L	T	R	L	T	R	L	T	R	L	T	R
Movement:	5	2	12	1	6	16	3	8	18	7	4	14
Volume, veh/h	200	1,000	10	194	968	10	100	500	50	100	500	50
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj. Factor (A_pbT)	1.000		1.000	1.000		1.000	1.000		1.000	1.000		1.000
Parking, Bus Adj. Factors (f_bb x f_p)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Downstream Lane Blockage Factor (f_ms)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Spillback Factor (f_sp)	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted Sat. Flow Rate, veh/h/ln	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
Lanes	1	2	1	1	2	1	1	2	0	1	2	0
Lane Assignment	L	T	R	L	T	R	L	TR	n.a.	L	TR	n.a.
Capacity, veh/h	236	1,856	789	233	1,848	785	217	617	61	217	617	61
Discharge Volume, veh/h	0	1,000	0	0	0	0	0	0	50	100	0	0
Proportion Arriving On Green	0.131	0.651	0.488	0.045	0.493	0.501	0.061	0.181	0.181	0.061	0.181	0.181
Approach Volume, veh/h		1,210			1,172			650			650	
Approach Delay, s/veh		18.0			23.4			39.7			39.7	
Approach Stop Rate, stops/veh		0.442			0.617			0.831			0.831	

With the exception of Initial Queue, Lanes, and Lane Assignment, the variables listed in Exhibit 30-32 have computed values. The volumes shown for the eastbound (EB), northbound (NB), and southbound (SB) movements are identical to the input volumes. The westbound (WB) volumes were computed from the input volumes during Step 1: Determine Traffic Demand Adjustments.

Specifically, they were reduced because the input westbound volume for this intersection exceeded the volume departing the upstream access point intersection (i.e., AP1).

Four factors are listed in the top half of Exhibit 30-32. These factors represent saturation flow rate adjustment factors. Their values are dependent on signal timing or lane volume, quantities that are computed during the iterative convergence loop (identified in the motorized vehicle methodology framework shown in Exhibit 18-8). As a result, the value of each factor also converges within this loop. The procedure for calculating the pedestrian–bicycle adjustment factor is described in Section 2 of Chapter 31. The procedure for calculating the parking–bus adjustment factor is described in Section 3 of Chapter 19. The procedure for calculating the downstream lane blockage (due to midsegment lane restriction) factor is described in Section 3 of this chapter. The methodology for calculating the spillback factor is described in Chapter 29.

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

Discharge volume is computed for movements that enter a segment during Step 1: Determine Traffic Demand Adjustments. At Signalized Intersection 1, the movements entering the segment are the eastbound through movement, the northbound right-turn movement, and the southbound left-turn movement. A value of 0.0 veh/h is shown for all other movements, which indicates that they are not relevant to this calculation. If volume exceeds capacity for any given movement, the discharge volume is set equal to the capacity. Otherwise, the discharge volume is equal to the movement volume.

The proportion arriving during green P is computed for internal movements during Step 3: Determine the Proportion Arriving During Green. In contrast, it is computed from the input platoon ratio for external movements.

The last three rows in Exhibit 30-32 represent summary statistics for the approach. The approach volume is the sum of the three movement volumes. Approach delay and approach stop rate are computed as volume-weighted averages for the lane groups served on an intersection approach.

Timer-Based Phase Data

Exhibit 30-33 provides a summary of the output data for Signalized Intersection 1 from a signal controller perspective. The controller has eight timing functions (or timers), with Timers 1 to 4 representing Ring 1 and Timers 5 to 8 representing Ring 2. The ring structure and phase assignments are described in Section 2 of Chapter 19. Timers 1, 2, 5, and 6 are used to control the east–west traffic movements on the segment. Timers 3, 4, 7, and 8 are used to control the north–south movements that cross the segment.

Exhibit 30-33
Example Problem 1: Timer-Based Phase Output Data

Timer Data

Timer:	1	2	3	4	5	6	7	8
	WB	EB	NB	SB	EB	WB	SB	NB
	L	T.R	L	T.T+R	L	T.R	L	T.T+R
Assigned Phase	1	2	3	4	5	6	7	8
Phase Duration (G+Y+Rc), s	15.90	52.84	9.13	22.13	16.10	52.63	9.13	22.13
Change Period (Y+Rc), s	3.00	4.00	3.00	4.00	3.00	4.00	3.00	4.00
Phase Start Time, s	35.27	51.16	4.00	13.14	35.27	51.37	4.00	13.14
Phase End Time, s	51.16	4.00	13.13	35.27	51.37	4.00	13.13	35.27
Max. Allowable Headway (MAH), s	3.13	0.00	3.13	3.06	3.13	0.00	3.13	3.06
Equivalent Maximum Green (Gmax), s	30.73	0.00	17.00	31.87	30.73	0.00	17.00	31.87
Max. Queue Clearance Time (g_c+I1), s	12.646	0.000	6.442	16.165	12.829	0.000	6.442	16.165
Green Extension Time (g_e), s	0.311	0.000	0.099	1.968	0.322	0.000	0.099	1.968
Probability of Phase Call (p_c)	0.995	0.000	0.938	1.000	0.996	0.000	0.938	1.000
Probability of Max Out (p_x)	0.000	0.000	0.000	0.016	0.000	0.000	0.000	0.016

Cycle Length, s: 100

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

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

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

The two signalized intersections in this example problem have lead–lead left-turn sequences. Hence, the timer number is equal to the phase number (e.g., the westbound movement is associated with Phase 1, which is assigned to Timer 1).

The phase duration shown in Exhibit 30-33 is the estimated average phase duration during the analysis period. It represents the sum of the green, yellow change, and red clearance intervals. For Timer 2 (i.e., Phase 2), the average green interval duration can be computed as 48.84 s (= 52.84 – 4.00).

The phase start time is the time the timer (and phase) starts, relative to system time 0.0. For Phase 2, the start time is 51.16 s. The end of the green interval associated with this phase is 100.0 s (= 51.16 + 48.84). This time is equal to the cycle length, so the end of green actually occurs at 0.0 s. This result is expected because Phase 2 is the coordinated phase and the offset to the end of Phase 2 (relative to system time 0.0) was input as 0.0 s.

The phase end time is the time the timer (and phase) ends relative to system time 0.0. For Phase 2, the end of the green interval occurs at 0.0 s and the end of the phase occurs 4.0 s later (i.e., the change period duration).

The remaining variables in Exhibit 30-33 apply to the noncoordinated phases (i.e., the actuated phases). These variables describe the phase timing and operation. They are described in more detail in Section 2 of Chapter 19 and Section 2 of Chapter 31.

Timer-Based Movement Data

Exhibit 30-34 summarizes the output for Signalized Intersection 1 as it relates to the movements assigned to each timer. Separate sections of output are shown in the exhibit for the left-turn, through, and right-turn movements. The assigned movement row identifies the movement (previously identified in Exhibit 30-32) assigned to each timer.

The saturation flow rate shown in Exhibit 30-34 is the saturation flow rate for the movement. The procedure for calculating these rates is described in Section 3 of Chapter 19 and Section 3 of Chapter 31. In general, the rate for a movement is the same as for a lane group when the lane group serves one movement. The rate is split between the movements when the lane group is shared by two or more movements.

Exhibit 30-34
Example Problem 1: Timer-Based Movement Output Data

Timer Data Timer:	1 WB L	2 EB T.R	3 NB L	4 SB T.T+R	5 EB L	6 WB T.R	7 SB L	8 NB T.T+R
Left-Turn Movement Data								
Assigned Movement	1		3		5		7	
Mvmt. Sat Flow, veh/h	1,805.00		1,805.00		1,805.00		1,805.00	
Through Movement Data								
Assigned Movement		2		4		6		8
Mvmt. Sat Flow, veh/h		3,800.00		3,401.19		3,800.00		3,401.19
Right-Turn Movement Data								
Assigned Movement		12		14		16		18
Mvmt. Sat Flow, veh/h		1,615.00		338.99		1,615.00		338.99

Timer-Based Lane Group Data

The motorized vehicle methodology described in Chapter 19 computes a variety of output statistics that portray the operation of each intersection lane group. The example problem in Chapter 19 illustrates these statistics and discusses their interpretation. The output data for the individual lane groups are not repeated in this chapter. Instead, the focus of the remaining discussion is on the access point output and the performance measures computed for the two through movements on the segment (i.e., eastbound through and westbound through).

Access Point Data

Exhibit 30-35 illustrates the output statistics for the two access point intersections located on the segment. The first six rows listed in the exhibit correspond to Access Point Intersection 1 (AP1), and the second six rows correspond to Access Point Intersection 2 (AP2). Additional sets of six rows would be provided in this table if additional access point intersections were evaluated.

Exhibit 30-35
Example Problem 1: Movement-Based Access Point Output Data

Access Point Data Segment 1 Movement:	EB L 1	EB T 2	EB R 3	WB L 4	WB T 5	WB R 6	NB L 7	NB T 8	NB R 9	SB L 10	SB T 11	SB R 12
Access Point Intersection No. 1												
1: Volume, veh/h	74.80	981.71	93.50	75.56	991.70	94.45	80.00	0.00	100.00	80.00	0.00	100.00
1: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1: Proportion time blocked	0.150			0.160			0.250	0.250	0.160	0.250	0.250	0.150
1: Delay to through vehicles, s/veh		0.193			0.194							
1: Prob. inside lane blocked by left		0.115			0.115							
1: Dist. from West/South signal, ft	600											
Access Point Intersection No. 2												
2: Volume, veh/h	75.56	991.70	94.45	74.80	981.71	93.50	80.00	0.00	100.00	80.00	0.00	100.00
2: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
2: Proportion time blocked	0.160			0.150			0.250	0.250	0.150	0.250	0.250	0.160
2: Delay to through vehicles, s/veh		0.194			0.193							
2: Prob. inside lane blocked by left		0.115			0.115							
2: Dist. from West/South signal, ft	1,200											

The eastbound and westbound volumes listed in Exhibit 30-35 are not equal to the input volumes. These volumes were adjusted during Step 1: Determine Traffic Demand Adjustments so that they equal the volume discharging from the upstream intersection. This routine achieves balance between all junction pairs (e.g., between Signalized Intersection 1 and Access Point Intersection 1, between Access Point Intersection 1 and Access Point Intersection 2, and so forth).

The "proportion of time blocked" is computed during Step 3: Determine the Proportion Arriving During Green. It represents the proportion of time during the cycle that the associated access point movement is blocked by the presence of a platoon passing through the intersection. For major-street left turns, the platoon of concern approaches from the opposing direction. For the minor-street left turn, platoons can approach from either direction and can combine to block this left turn for extended time periods. This trend can be seen by comparing the proportion of time blocked for the eastbound (major-street) left turn (i.e., 0.15) with that for the northbound (minor-street) left turn (i.e., 0.25) at Access Point Intersection 1.

The "delay to through vehicles" is computed during Step 2: Determine Running Time. It represents the sum of the delay due to vehicles turning left from the major street and the delay due to vehicles turning right from the major street. This delay tends to be small compared with typical signalized intersection delay values. But it can reduce overall travel speed if there are several high-volume access points on a street and only one or two through lanes in each direction of travel.

The "probability of the inside through lane being blocked" is also computed during Step 2: Determine Running Time as part of the delay-to-through-vehicles procedure. This variable indicates the probability that the left-turn bay at an access point will overflow into the inside through lane on the street segment. Hence, it indicates the potential for a through vehicle to be delayed by a left-turn maneuver. The segment being evaluated has an undivided cross section, and no left-turn bays are provided at the access point intersections. In this situation, the probability of overflow is 0.115, indicating that the inside lane is blocked about 11.5% of the time.

Results

Exhibit 30-36 summarizes the performance measures for the segment. Also shown are the results from the spillback check conducted during Step 1: Determine Traffic Demand Adjustments. The movements indicated in the column heading are those exiting the segment at a boundary intersection. Thus, the westbound movements on Segment 1 are those occurring at Signalized Intersection 1. Similarly, the eastbound movements on Segment 1 are those occurring at Signalized Intersection 2.

Segment Summary		EB	EB	EB	WB	WB	WB
		L	T	R	L	T	R
Seg.No.	Movement:	5	2	12	1	6	16
1	Bay/Lane Spillback Time, h	never	never	never	never	never	never
1	ShrdLane Spillback Time, h	never	never	never	never	never	never
1	Base Free-Flow Speed, mph		40.78			40.78	
1	Running Time, s		33.54			33.54	
1	Running Speed, mph		36.59			36.59	
1	Through Delay, s/veh		18.310			18.310	
1	Travel Speed, mph		23.67			23.67	
1	Stop Rate, stops/veh		0.547			0.547	
1	Spatial Stop Rate, stops/mi		1.61			1.61	
1	Through vol/cap ratio		0.52			0.52	
1	Level of Service		C			C	
1	Proportion Left Lanes		0.33			0.33	
1	Auto. Traveler Perception Score		2.53			2.53	
SPILLBACK TIME, h: never							

Exhibit 30-36
Example Problem 1:
Performance Measure Summary

The spillback check procedure computes the time of spillback for each of the internal movements. For turn movements, the bay/lane spillback time is the time before the turn bay overflows. For through movements, the bay/lane spillback time is the time before the through lane overflows due only to through demand. If a turn bay exists and it overflows, the turn volume will queue in the adjacent through lane. For this scenario, the shared lane spillback time is computed and used instead of the bay/lane spillback time. If several movements experience spillback, the time of first spillback is reported at the bottom of Exhibit 30-36.

The output data for the two through movements are listed in Exhibit 30-36, starting with the third row. The base free-flow speed (FFS) and running time statistics are computed during Step 2: Determine Running Time. The through delay listed is computed during Step 5: Determine Through Control Delay. It is a weighted average delay for the lane groups serving through movements at the downstream boundary intersection. The weight used in this average is the volume of through vehicles served by the lane group.

The base free-flow speed is 40.78 mi/h. By interpolating this value between those in Exhibit 18-1, the threshold travel speeds for LOS A, B, C, D, and E are as follows: >32.6, >27.5, >20.5, >16.3, and >12.3 mi/h, respectively. Thus, the travel speed for the eastbound direction of 23.67 mi/h corresponds to LOS C. The same conclusion is reached for the westbound travel direction.

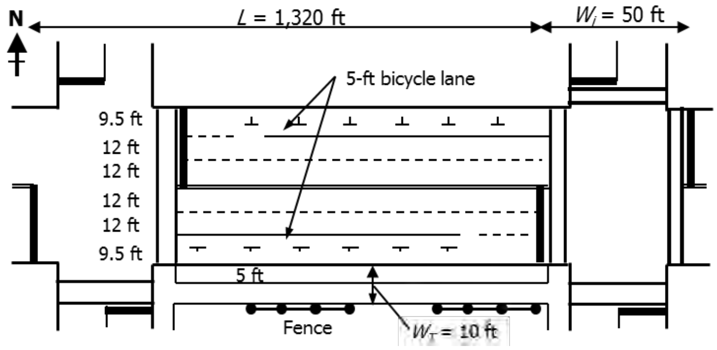
Each travel direction has one left-turn bay and three intersections. Thus, the proportion of intersections with left-turn lanes is 0.33. This proportion is used in Step 10: Determine Automobile Traveler Perception Score to compute the score of 2.53, which suggests that most automobile travelers would find segment service to be very good.

EXAMPLE PROBLEM 2: PEDESTRIAN LOS

The Segment

The sidewalk of interest is located along a 1,320-ft urban street segment. The segment is part of a collector street located near a community college. It is shown in Exhibit 30-37. Sidewalk is only shown for the south side of the segment for the convenience of illustration. It also exists on the north side of the segment.

Exhibit 30-37
Example Problem 2: Segment Geometry



The Question

What is the pedestrian LOS for the sidewalk on the south side of the segment?

The Facts

The geometric details of the sidewalk and street cross section are shown in Exhibit 30-37. Both boundary intersections are signalized. Crossing the segment at uncontrolled midsegment locations is legal. The following additional information is known about the sidewalk and street segment:

Traffic characteristics:

Midsegment flow rate in eastbound direction: 940 veh/h

Pedestrian flow rate in south sidewalk (walking in both directions): 2,000 p/h

Proportion of on-street parking occupied during analysis period: 0.20

Geometric characteristics:

Outside shoulder width: none

Parking lane width: 9.5 ft

Cross section has raised curb along outside edge of roadway

Effective width of fixed objects on sidewalk: 0.0 ft (no objects present)

Presence of trees, bushes, or other vertical objects in buffer: No

Other data:

Pedestrians can cross the segment legally and do so somewhat uniformly along its length

Proportion of sidewalk adjacent to window display: 0.0

Proportion of sidewalk adjacent to building face: 0.0

Proportion of sidewalk adjacent to fence: 0.50

Performance measures obtained from supporting methodologies:

Motorized vehicle running speed: 33 mi/h

Pedestrian delay when walking parallel to the segment: 40 s/p

Pedestrian delay when crossing the segment at the nearest signal-controlled crossing: 80 s/p

Pedestrian delay crossing the segment at an uncontrolled midsegment location: 740 s/p

Pedestrian LOS score for the downstream intersection: 3.60

Outline of Solution

First, the pedestrian space will be calculated for the sidewalk. This measure will then be compared with the qualitative descriptions of pedestrian space listed in Exhibit 18-15. Next, the pedestrian travel speed along the sidewalk will be calculated. Finally, LOS for the segment will be determined by using the computed pedestrian LOS score and the pedestrian space variables.

Computational Steps

Step 1: Determine Free-Flow Walking Speed

The average free-flow walking speed is estimated to be 4.4 ft/s on the basis of the guidance provided.

Step 2: Determine Average Pedestrian Space

The shy distance on the inside of the sidewalk is computed with Equation 18-24.

$$W_{s,i} = \max(W_{buf}, 1.5)$$

$$W_{s,i} = \max\,(5.0, 1.5)$$

$$W_{s,i} = 5.0\text{ ft}$$

The shy distance on the outside of the sidewalk is computed with Equation 18-25.

$$W_{s,o} = 3.0\,p_{\text{window}} + 2.0\,p_{\text{building}} + 1.5\,p_{\text{fence}}$$

$$W_{s,o} = 3.0(0.0) + 2.0(0.0) + 1.5(0.50)$$

$$W_{s,o} = 0.75\text{ ft}$$

There are no fixed objects present on the sidewalk, so the adjusted fixed-object effective widths for the inside and outside of the sidewalk are both equal to 0.0 ft. The effective sidewalk width is computed with Equation 18-23.

$$W_E = W_T - W_{O,i} - W_{O,o} - W_{s,i} - W_{s,o} \geq 0.0$$

$$W_E = 10 - 0.0 - 0.0 - 5.0 - 0.75$$

$$W_E = 4.25\text{ ft}$$

The pedestrian flow per unit width of sidewalk is computed with Equation 18-28 for the subject sidewalk.

$$v_p = \frac{v_{ped}}{60\,W_E}$$

$$v_p = \frac{2{,}000}{60(4.25)}$$

$$v_p = 7.84\text{ p/ft/min}$$

The average walking speed S_p is computed with Equation 18-29.

$$S_p = (1 - 0.00078\,v_p^2)\,S_{pf} \geq 0.5\,S_{pf}$$

$$S_p = [1 - 0.00078(7.84)^2](4.4)$$

$$S_p = 4.19\text{ ft/s}$$

Finally, Equation 18-30 is used to compute average pedestrian space.

$$A_p = 60\frac{S_p}{v_p}$$

$$A_p = 60\frac{4.19}{7.84}$$

$$A_p = 32.0\ \text{ft}^2/\text{p}$$

The pedestrian space can be compared with the ranges provided in Exhibit 18-15 to make some judgments about the performance of the subject intersection corner. The criteria for platoon flow are considered applicable given the influence of the signalized intersections. According to the qualitative descriptions provided in this exhibit, walking speed will be restricted, as will the ability to pass slower pedestrians.

Step 3: Determine Pedestrian Delay at Intersection

The pedestrian methodology in Chapter 19, Signalized Intersections, was used to estimate two pedestrian delay values. One is the delay at the boundary intersection experienced by a pedestrian walking parallel to segment d_{pp}. This delay was computed to be 40 s/p. The second is the delay experienced by a pedestrian crossing the segment at the nearest signal-controlled crossing d_{pc}. This delay was computed to be 80 s/p.

The pedestrian methodology in Chapter 20, Two-Way STOP-Controlled Intersections, was used to estimate the delay incurred while waiting for an acceptable gap in traffic d_{pw} at a midsegment location. As given in The Facts, this delay was computed to be 740 s/p.

Step 4: Determine Pedestrian Travel Speed

The pedestrian travel speed is computed with Equation 18-31.

$$S_{Tp,seg} = \frac{L}{\frac{L}{S_p} + d_{pp}}$$

$$S_{Tp,seg} = \frac{1{,}320}{\frac{1{,}320}{4.19} + 40}$$

$$S_{Tp,seg} = 3.72\ \text{ft/s}$$

This walking speed is slightly less than 4.0 ft/s and is considered acceptable, but a higher speed is desirable.

Step 5: Determine Pedestrian LOS Score for Intersection

The pedestrian methodology in Chapter 19 was used to determine the pedestrian LOS score for the downstream boundary intersection $I_{p,int}$. As given in The Facts, it was computed to be 3.60.

Step 6: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link is computed from three factors. However, before these factors can be calculated, several cross-section variables need to be adjusted and several coefficients need to be calculated. These variables and coefficients are calculated first. Then, the three factors are computed. Finally, they are combined to determine the desired score.

The midsegment demand flow rate is greater than 160 veh/h. The street cross section is curbed but there is no shoulder, so the adjusted width of paved outside shoulder W_{os}^* is 0.0 ft. Therefore, the effective total width of the outside through lane, bicycle lane, and shoulder W_v is computed as

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

$$W_v = 12 + 5 + 0 + 9.5$$

$$W_v = 26.5 \text{ ft}$$

Because the proportion of occupied on-street parking is less than 0.25 and the sum of the bicycle lane and parking lane widths exceeds 10.0 ft, the effective width of the combined bicycle lane and parking lane W_l is set to 10.0 ft.

The adjusted available sidewalk width W_{aA} is computed as

$$W_{aA} = \min\,(W_T - W_{buf}, 10)$$

$$W_{aA} = \min\,(10 - 5, 10)$$

$$W_{aA} = 5 \text{ ft}$$

The sidewalk width coefficient f_{sw} is computed as

$$f_{sw} = 6.0 - 0.3\,W_{aA}$$

$$f_{sw} = 6.0 - 0.3(5.0)$$

$$f_{sw} = 4.5 \text{ ft}$$

The buffer area coefficient f_b is equal to 1.0 because there is no continuous barrier at least 3.0 ft high located in the buffer area.

The motorized vehicle methodology described in Section 3 of Chapter 18 was used to determine the motorized vehicle running speed S_R for the subject segment. This speed was computed to be 33.0 mi/h.

The cross-section adjustment factor is computed with Equation 18-33.

$$F_w = -1.2276 \ln\,(W_v + 0.5\,W_l + 50\,p_{pk} + W_{buf} f_b + W_{aA} f_{sw})$$

$$F_w = -1.2276 \ln\,[26.5 + 0.5(10) + 50(0.20) + 5.0(1.0) + 5.0(4.5)]$$

$$F_w = -5.20$$

The motorized vehicle volume adjustment factor is computed with Equation 18-34.

$$F_v = 0.0091 \frac{v_m}{4\,N_{th}}$$

$$F_v = 0.0091 \frac{940}{4(2)}$$

$$F_v = 1.07$$

The motorized vehicle speed adjustment factor is computed with Equation 18-35.

$$F_s = 4\left(\frac{S_R}{100}\right)^2$$

$$F_s = 4\left(\frac{33.0}{100}\right)^2$$

$$F_s = 0.44$$

Finally, the pedestrian LOS score for the link $I_{p,\text{link}}$ is calculated with Equation 18-32.

$$I_{p,\text{link}} = 6.0468 + F_w + F_v + F_s$$

$$I_{p,\text{link}} = 6.0468 + (-5.20) + 1.07 + 0.44$$

$$I_{p,\text{link}} = 2.35$$

Step 7: Determine Link LOS

The pedestrian LOS for the link is determined by using the pedestrian LOS score from Step 6. This score is compared with the link-based pedestrian LOS thresholds on the right side of Exhibit 18-2 to determine that the LOS for the specified direction of travel along the subject link is B.

Step 8: Determine Roadway Crossing Difficulty Factor

Crossings occur somewhat uniformly along the length of the segment, and the segment is bounded by two signalized intersections. Thus, the distance D_c is assumed to equal one-third of the segment length, or 440 ft (= 1,320/3), and the diversion distance D_d is computed as 880 ft (= 2 × 440 ft).

The delay incurred due to diversion is calculated by using Equation 18-37.

$$d_{pd,LOS} = 0.084\frac{2D_d}{S_p} + d_{pc}$$

$$d_{pd,LOS} = 0.084\frac{2 \times 880}{4.19} + 80$$

$$d_{pd,LOS} = 121 \text{ s/p}$$

Both the perceived diversion delay $d_{pd,LOS}$ of 121 s/p and the delay waiting for an adequate gap d_{pw} of 740 s/p are greater than 70 s and therefore, from Exhibit 18-20A, the LOS scores associated with each delay equal 6. Therefore, based on Equation 18-38, the midsegment crossing LOS score is 6:

$$I_{p,mx} = \min\left[I_{pw}, I_{pd}, 6\right] = \min[6, 6, 6] = 6$$

Step 9: Determine Pedestrian LOS Score for Segment

Equation 18-39 is used to determine the pedestrian LOS score for the segment. The proportion of pedestrian demand desiring to cross midblock p_{mx} is assumed to be the default value of 0.35.

$$I_{p,seg} = \left[\frac{\left(I_{p,link}\,[1 - p_{mx}] + I_{p,mx}\,p_{mx}\right)^3 L/S_p + \left(I_{p,int}\right)^3 d_{pp}}{L/S_p + d_{pp}}\right]^{1/3}$$

$$I_{p,seg} = \left[\frac{(2.35\,[1 - 0.35] + 6.0 \times\ 0.35)^3 \left(\frac{1{,}320}{4.19}\right) + (3.60)^3 \times 40}{\left(\frac{1{,}320}{4.19}\right) + 40}\right]^{1/3}$$

$$I_{p,seg} = 3.62$$

Step 10: Determine Segment LOS

The pedestrian LOS for the segment is determined by using the pedestrian LOS score from Step 9 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds on the left side of Exhibit 18-2 to determine that the LOS for the specified direction of travel along the subject segment is D.

EXAMPLE PROBLEM 3: BICYCLE LOS

The Segment

The bicycle lane of interest is located along a 1,320-ft urban street segment. The segment is part of a collector street located near a community college. The bicycle lane is provided for the eastbound direction of travel, as shown in Exhibit 30-38.

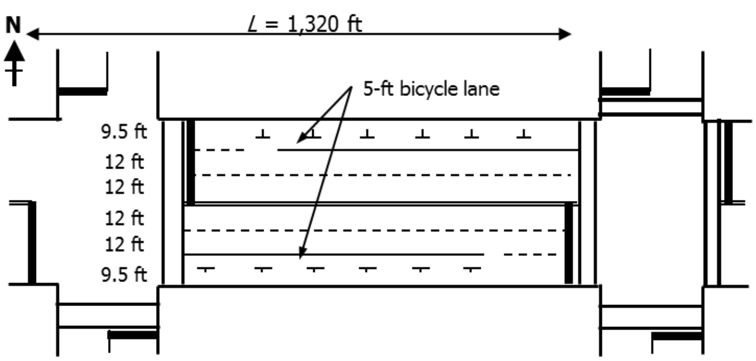


Exhibit 30-38
Example Problem 3: Segment Geometry

The Question

What is the bicycle LOS for the eastbound bicycle lane?

The Facts

The geometric details of the street cross section are shown in Exhibit 30-38. Both boundary intersections are signalized. The following additional information is known about the street segment:

Traffic characteristics:

Midsegment flow rate in eastbound direction: 940 veh/h

Percent heavy vehicles: 8.0%

Proportion of on-street parking occupied during analysis period: 0.20

Geometric characteristics:

Outside shoulder width: none

Parking lane width: 9.5 ft

Median type: undivided

Cross section has raised curb along the outside edge of the roadway

Number of access point approaches on right side of segment in subject travel direction: 3

Other data:

Pavement condition rating: 2.0

Performance measures obtained from supporting methodologies:

Motorized vehicle running speed: 33 mi/h

Bicycle control delay: 40 s/bicycle

Bicycle LOS score for the downstream intersection: 0.08

Outline of Solution

First, the bicycle delay at the boundary intersection will be computed. This delay will then be used to compute the bicycle travel speed. Next, a bicycle LOS score will be computed for the link. It will then be combined with a similar score for the boundary intersection and used to compute the bicycle LOS score for the segment. Finally, LOS for the segment will be determined by using the computed score and the thresholds in Exhibit 18-3.

Computational Steps

Step 1: Determine Bicycle Running Speed

The average bicycle running speed S_b could not be determined from field data. Therefore, it was estimated to be 15 mi/h on the basis of the guidance provided.

Step 2: Determine Bicycle Delay at Intersection

The motorized vehicle methodology in Chapter 19, Signalized Intersections, was used to estimate the bicycle delay at the boundary intersection d_b. This delay was computed to be 40.0 s/bicycle.

Step 3: Determine Bicycle Travel Speed

The segment running time of through bicycles is computed as

$$t_{Rb} = \frac{3{,}600\ L}{5{,}280\ S_b}$$

$$t_{Rb} = \frac{3{,}600(1{,}320)}{5{,}280(15)}$$

$$t_{Rb} = 60.0 \text{ s}$$

The average bicycle travel speed is computed with Equation 18-40.

$$S_{Tb,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rb} + d_b)}$$

$$S_{Tb,seg} = \frac{3{,}600(1{,}320)}{5{,}280\ (60.0 + 40.0)}$$

$$S_{Tb,seg} = 9.0\ \text{mi/h}$$

This travel speed is adequate, but a speed of 10 mi/h or more is considered desirable.

Step 4: Determine Bicycle LOS Score for Intersection

The bicycle methodology in Chapter 19 was used to determine the bicycle LOS score for the boundary intersection $I_{b,int}$. It was computed to be 0.08.

Step 5: Determine Bicycle LOS Score for Link

The bicycle LOS score is computed from four factors. However, before these factors can be calculated, several cross-section variables need to be adjusted. These variables are calculated first, and then the four factors are computed. Finally, they are combined to determine the desired score.

The street cross section is curbed but there is no shoulder, so the adjusted width of the paved outside shoulder W_{os}^* is 0.0 ft. Therefore, the total width of the outside through lane, bicycle lane, and paved shoulder W_t is computed as

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

$$W_t = 12 + 5 + 0$$

$$W_t = 17\ \text{ft}$$

The variable W_t does not include the width of the parking lane in this instance because the proportion of occupied on-street parking exceeds 0.0.

The total width of shoulder, bicycle lane, and parking lane W_l is computed as

$$W_l = W_{bl} + W_{os}^* + W_{pk}$$

$$W_l = 5 + 0 + 9.5$$

$$W_l = 14.5\ \text{ft}$$

The midsegment demand flow rate is greater than 160 veh/h. Therefore, the effective total width of the outside through lane, bicycle lane, and shoulder as a function of traffic volume W_v is equal to W_t.

The total width of shoulder, bicycle lane, and parking lane W_l exceeds 4.0 ft. Therefore, the effective width of the outside through lane is computed as

$$W_e = W_v + W_l - 20\ p_{pk} \geq 0.0$$

$$W_e = 17 + 14.5 - 20(0.20) \geq 0.0$$

$$W_e = 27.5\ \text{ft}$$

The percent heavy vehicles is less than 50%, so the adjusted percent heavy vehicles P_{HVa} is equal to the input percent heavy vehicles P_{HV} of 8.0%.

The motorized vehicle methodology described in Section 3 of Chapter 18 was used to determine the motorized vehicle running speed S_R for the subject segment. This speed was computed to be 33.0 mi/h, which exceeds 21 mi/h. Therefore, the adjusted motorized vehicle speed S_{Ra} is also equal to 33.0 mi/h.

The midsegment demand flow rate is greater than 8 veh/h (= 4 N_{th}), so the adjusted midsegment demand flow rate v_{ma} is equal to the input demand flow rate of 940 veh/h.

The cross-section adjustment factor is computed with Equation 18-42.

$$F_w = -0.005\, W_e^2$$

$$F_w = -0.005(27.5)^2$$

$$F_w = -3.78$$

The motorized vehicle volume adjustment factor comes from Equation 18-43.

$$F_v = 0.507 \ln\left(\frac{v_{ma}}{4\, N_{th}}\right)$$

$$F_v = 0.507 \ln\left(\frac{940}{4(2)}\right)$$

$$F_v = 2.42$$

The motorized vehicle speed adjustment factor is computed with Equation 18-44.

$$F_S = 0.199[1.1199 \ln(S_{Ra} - 20) + 0.8103](1 + 0.1038 P_{HVa})^2$$

$$F_S = 0.199[1.1199 \ln(33.0 - 20) + 0.8103][1 + 0.1038(8.0)]^2$$

$$F_S = 2.46$$

The pavement condition adjustment factor is computed with Equation 18-45.

$$F_p = \frac{7.066}{P_c^2}$$

$$F_p = \frac{7.066}{(2.0)^2}$$

$$F_p = 1.77$$

Finally, the bicycle LOS score for the link $I_{b,\text{link}}$ is calculated with Equation 18-41.

$$I_{b,\text{link}} = 0.760 + F_w + F_v + F_S + F_p$$

$$I_{b,\text{link}} = 0.760 - 3.78 + 2.42 + 2.46 + 1.77$$

$$I_{b,\text{link}} = 3.62$$

Step 6: Determine Link LOS

The bicycle LOS for the link is determined by using the bicycle LOS score from Step 5. This score is compared with the link-based bicycle LOS thresholds in Exhibit 18-3 to determine that the LOS for the specified direction of travel along the subject link is D.

Step 7: Determine Bicycle LOS Score for Segment

The unsignalized conflicts factor is computed with Equation 18-47.

$$F_c = 0.035\left(\frac{5{,}280\ N_{ap,s}}{L} - 20\right)$$

$$F_c = 0.035\left[\frac{5{,}280\ (3)}{1{,}320} - 20\right]$$

$$F_c = -0.28$$

The bicycle LOS score for the segment is computed with Equation 18-46.

$$I_{b,seg} = 0.75\left[\frac{\left(F_c + I_{b,\mathrm{link}} + 1\right)^3 t_{R,b} + \left(I_{b,int} + 1\right)^3 d_b}{t_{R,b} + d_b}\right]^{\frac{1}{3}} + 0.125$$

$$I_{b,seg} = 0.75\left[\frac{[(-0.28) + 3.62 + 1]^3(60) + (0.08 + 1)^3(40)}{60 + 40}\right]^{\frac{1}{3}} + 0.125$$

$$I_{b,seg} = 2.88$$

Step 8: Determine Segment LOS

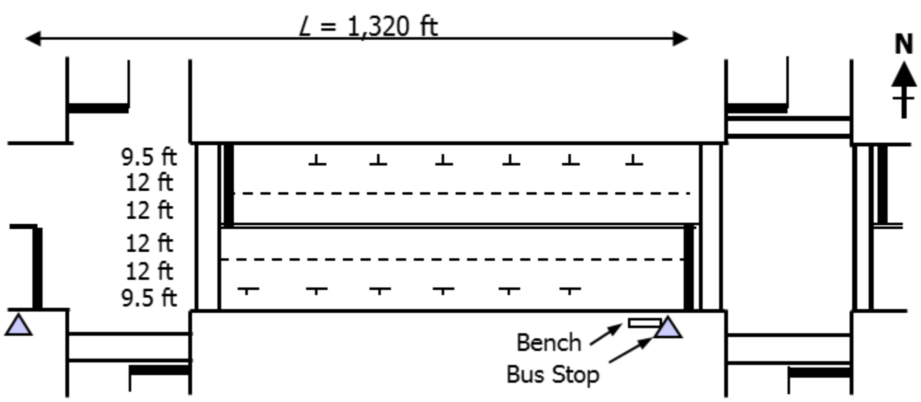
The bicycle LOS for the segment is determined by using the bicycle LOS score from Step 7. This score is compared with the segment-based bicycle LOS thresholds in Exhibit 18-3 to determine that the LOS for the specified direction of travel along the subject segment is C.

EXAMPLE PROBLEM 4: TRANSIT LOS

The Segment

The transit route of interest travels east along a 1,320-ft urban street segment. The segment is part of a collector street located near a community college. It is shown in Exhibit 30-39. A bus stop is provided on the south side of the segment for the subject route.

Exhibit 30-39
Example Problem 4: Segment Geometry



The Question

What is the transit LOS for the eastbound bus route on the subject segment?

The Facts

The geometric details of the segment are shown in Exhibit 30-39. Both boundary intersections are signalized. There is one stop in the segment for the eastbound route. The following additional information is known about the bus stop and street segment:

Transit characteristics:

Dwell time: 20.0 s

Transit frequency: 4 veh/h

Excess wait time data are not available for the stop, but the on-time performance of the route (based on a standard of up to 5 min late being considered "on time") at the previous time point is known (92%)

Passenger load factor: 0.83 passengers/seat

Other data:

Area type: not in a central business district

g/*C* ratio at downstream boundary intersection: 0.4729

Cycle length: 140 s

The bus stop in the segment has a bench, but no shelter

Number of routes serving the segment: 1

The bus stop is accessed from the right-turn lane (i.e., the stop is off-line). Buses are exempt from the requirement to turn right but have no other traffic priority

Performance measures obtained from supporting methodologies:

Motorized vehicle running speed: 33 mi/h

Pedestrian LOS score for the link: 3.53

Through vehicle control delay at the downstream boundary intersection: 19.4 s/veh

Reentry delay: 16.17 s

Outline of Solution

First, the transit vehicle segment running time will be computed. Next, the control delay at the boundary intersection will be obtained and used to compute the transit vehicle segment travel speed. Then the transit wait–ride score will be computed. This score will be combined with the pedestrian LOS score for the link to compute the transit LOS score for the segment. Finally, LOS for the segment will be determined by comparing the computed score with the thresholds identified in Exhibit 18-3.

Computational Steps

Step 1: Determine Transit Vehicle Running Time

The transit vehicle running time is based on the segment running speed and delay due to a transit vehicle stop. These components are calculated first, and then running time is calculated.

Transit vehicle segment running speed can be computed with Equation 18-48.

$$S_{Rt} = \min\left(S_R, \frac{61}{1 + e^{-1.00+(1{,}185\ N_{ts}/L)}}\right)$$

$$S_{Rt} = \min\left(33.0, \frac{61}{1 + e^{-1.00+(1{,}185(1)/1{,}320)}}\right)$$

$$S_{Rt} = 32.1 \text{ mi/h}$$

The acceleration and deceleration rates are unknown, so they are assumed to be 3.3 ft/s² and 4.0 ft/s², respectively, on the basis of data given in the *Transit Capacity and Quality of Service Manual* (*9*).

The bus stop is located on the near side of a signalized intersection. From Equation 18-50, the average proportion of bus stop acceleration–deceleration delay not due to the intersection's traffic control f_{ad} is equal to the g/C ratio for the through movement in the bus's direction of travel (in this case, eastbound). The effective green time g is 66.21 s (calculated as the phase duration minus the change period), and the cycle length is 140 s. Therefore, f_{ad} is 0.4729.

Equation 18-49 can now be used to compute the portion of bus stop delay due to acceleration and deceleration.

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{S_{Rt}}{2}\right)\left(\frac{1}{r_{at}} + \frac{1}{r_{dt}}\right)f_{ad}$$

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{32.1}{2}\right)\left(\frac{1}{3.3} + \frac{1}{4.0}\right)(0.4729)$$

$$d_{ad} = 6.15 \text{ s}$$

Equation 18-51 is used to compute the portion of bus stop delay due to serving passengers. The input average dwell time of 20.0 s and an f_{dt} value of 0.4729 are used in the equation, on the basis of the stop's near-side location at a traffic signal and the g/C ratio computed in a previous step. The f_{dt} factor is used to avoid double-counting the portion of passenger service time that occurs during the signal's red indication and is therefore included as part of control delay.

$$d_{ps} = t_d f_{dt}$$

$$d_{ps} = (20.0)(0.4729)$$

$$d_{ps} = 9.46 \text{ s}$$

The bus stop is located in the right-turn lane; therefore, the bus is subject to reentry delay on leaving the stop. On the basis of the guidance for reentry delay for a near-side stop at a traffic signal, the reentry delay d_{re} is equal to the queue service time g_s. This time is calculated to be 16.17 s by following the procedures in Section 3 of Chapter 31, Signalized Intersections: Supplemental.

Equation 18-52 is used to compute the total delay due to the transit stop.

$$d_{ts} = d_{ad} + d_{ps} + d_{re}$$

$$d_{ts} = 6.15 + 9.46 + 16.17$$

$$d_{ts} = 31.78 \text{ s}$$

Equation 18-53 is used to compute transit vehicle running time on the basis of the previously computed components.

$$t_{Rt} = \frac{3{,}600\ L}{5{,}280\ S_{Rt}} + \sum_{i=1}^{N_{ts}} d_{ts,i}$$

$$t_{Rt} = \frac{3{,}600(1{,}320)}{5{,}280(32.1)} + 31.78$$

$$t_{Rt} = 59.9\ \mathrm{s}$$

Step 2: Determine Delay at Intersection

The through delay d_t at the boundary intersection is set equal to the through vehicle control delay exiting the segment at this intersection. The latter delay is 19.4 s/veh. Thus, the through delay d_t is equal to 19.4 s/veh.

Step 3: Determine Travel Speed

The average transit travel speed is computed with Equation 18-55.

$$S_{Tt,seg} = \frac{3{,}600\ L}{5{,}280\ (t_{Rt} + d)}$$

$$S_{Tt,seg} = \frac{3{,}600(1{,}320)}{5{,}280(59.9 + 19.4)}$$

$$S_{Tt,seg} = 11.3\ \mathrm{mi/h}$$

Step 4: Determine Transit Wait–Ride Score

The wait–ride score is based on the headway factor and the perceived travel time factor. Each of these components is calculated separately. The wait–ride score is then calculated.

The input data indicate that there is one route on the segment, and its frequency is 4 veh/h. The headway factor is computed with Equation 18-56.

$$F_h = 4.00e^{-1.434/(v_s+0.001)}$$

$$F_h = 4.00e^{-1.434/(4+0.001)}$$

$$F_h = 2.80$$

The perceived travel time factor is based on several intermediate variables that need to be calculated first. The first of these calculations is the amenity time rate. It is calculated by using Equation 18-60. A default passenger trip length of 3.7 mi is used in the absence of other information.

$$T_{at} = \frac{1.3\ p_{sh} + 0.2\ p_{be}}{L_{pt}}$$

$$T_{at} = \frac{1.3(0.0) + 0.2(1.0)}{3.7}$$

$$T_{at} = 0.054\ \mathrm{min/mi}$$

Since no information is available for actual excess wait time but on-time performance information is available for the route, Equation 18-61 is used to estimate excess wait time.

$$t_{ex} = [t_{late}(1 - p_{ot})]^2$$

$$t_{ex} = [5.0(1 - 0.92)]^2$$

$$t_{ex} = 0.16 \text{ min}$$

The excess wait time rate T_{ex} is then the excess wait time t_{ex} divided by the average passenger trip length L_{pt}: 0.16/3.7 = 0.043 min/mi.

The passenger load waiting factor is computed with Equation 18-59.

$$a_1 = 1 + \frac{4\,(F_l - 0.80)}{4.2}$$

$$a_1 = 1 + \frac{4\,(0.83 - 0.80)}{4.2}$$

$$a_1 = 1.03$$

The perceived travel time rate is computed with Equation 18-58.

$$T_{ptt} = \left(a_1 \frac{60}{S_{Tt,seg}}\right) + (2\,T_{ex}) - T_{at}$$

$$T_{ptt} = \left(1.03 \frac{60}{11.3}\right) + [2(0.043)] - 0.054$$

$$T_{ptt} = 5.50 \text{ min/mi}$$

The segment is not located in a central business district of a metropolitan area with a population of 5 million or more, so the base travel time rate T_{btt} is equal to 4.0 min/mi. The perceived travel time factor is computed with Equation 18-57.

$$F_{tt} = \frac{(e-1)\,T_{btt} - (e+1)\,T_{ptt}}{(e-1)\,T_{ptt} - (e+1)\,T_{btt}}$$

$$F_{tt} = \frac{(-0.40-1)(4.0) - (-0.40+1)(5.50)}{(-0.40-1)(5.50) - (-0.40+1)(4.0)}$$

$$F_{tt} = 0.881$$

Finally, the transit wait–ride score is computed with Equation 18-62.

$$s_{w\text{-}r} = F_h F_{tt}$$

$$s_{w\text{-}r} = (2.80)(0.883)$$

$$s_{w\text{-}r} = 2.47$$

Step 5: Determine Pedestrian LOS Score for Link

The pedestrian methodology described in Chapter 18 was used to determine the pedestrian LOS score for the link $I_{p,\text{link}}$. This score was computed to be 3.53.

Step 6: Determine Transit LOS Score for Segment

The transit LOS score for the segment is computed with Equation 18-63.

$$I_{t,seg} = 6.0 - 1.50\, s_{w\text{-}r} + 0.15\, I_{p,\text{link}}$$

$$I_{t,seg} = 6.0 - 1.50(2.47) + 0.15(3.53)$$

$$I_{t,seg} = 2.83$$

Step 7: Determine LOS

The transit LOS is determined by using the transit LOS score from Step 6. This performance measure is compared with the thresholds in Exhibit 18-3 to determine that the LOS for the specified bus route is C.

9. ROUNDABOUT SEGMENT METHODOLOGY

SCOPE OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of the motor vehicle mode on an urban street segment bounded by one or more roundabouts. The methodology is based on national research that measured the travel time performance of nine facilities containing three or more roundabouts in series (*10*). The methodology is designed to be integrated into the general motorized vehicle methodology for urban street segments described in Chapter 18. Only the relevant deviations from the general methodology are provided in this subsection.

LIMITATIONS OF THE METHODOLOGY

The methodologies in this subsection are based on regression analyses of field-measured data. The limits of these field data are provided in Exhibit 30-40. The analyst is cautioned with regard to the validity of the results when an input or intermediate calculated value is outside the range of the research data. In addition, the methodology does not account for capacity constraint caused by oversaturated conditions or the possible effects of an upstream signal on a downstream roundabout.

Exhibit 30-40
Validity Range of Inputs and Calculated Values for Analysis of Motor Vehicles on an Urban Street Roundabout Segment

Input or Calculated Value	Minimum	Maximum
Input Data		
Inscribed circle diameter (ft)	84	245
Number of circulating lanes	1	2
Segment length (ft)	540	7,900
Posted speed limit (mi/h)	25	50
Intermediate Calculations		
Central island diameter (ft)	48	187
Length of first portion of segment (ft)	270	3,953
Length of second portion of segment (ft)	244	3,993
Free-flow speed (mi/h)	26	53
Roundabout influence area for first portion of segment (ft)	235	1,446
Roundabout influence area for second portion of segment (ft)	73	897
Geometric delay for first portion of segment (s)	0.1	9.5
Geometric delay for second portion of segment (s)	0.1	6.6

REQUIRED INPUT DATA AND SOURCES

Exhibit 30-41 lists the additional required input data, potential data sources, and suggested default values for applying the methodology in this subsection. The reader should refer to Chapter 18 for a complete list of required input data. Guidance on selecting values for inscribed circle diameter and width of circulating lanes can be obtained elsewhere (*11*).

Exhibit 30-41
Additional Required Input Data, Potential Data Sources, and Default Values for Analysis of Motor Vehicles on an Urban Street Roundabout Segment

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Design Data	
Inscribed circle diameter of upstream and downstream roundabout (ft)	Field data, aerial photo, preliminary design	130 ft for one-lane roundabout 180 ft for two-lane roundabout
Number of circulating lanes of upstream and downstream roundabout (ft)	Field data, aerial photo, preliminary design	Must be provided
Average width of circulating lanes of upstream and downstream roundabout (ft)	Field data, aerial photo, preliminary design	20 ft for one-lane roundabout 15 ft for two-lane roundabout
	Performance Measure Data	
Control delay by lane at boundary roundabout (s/veh)	HCM method output	Must be provided
Capacity by lane at boundary roundabout (veh/h)	HCM method output	Must be provided

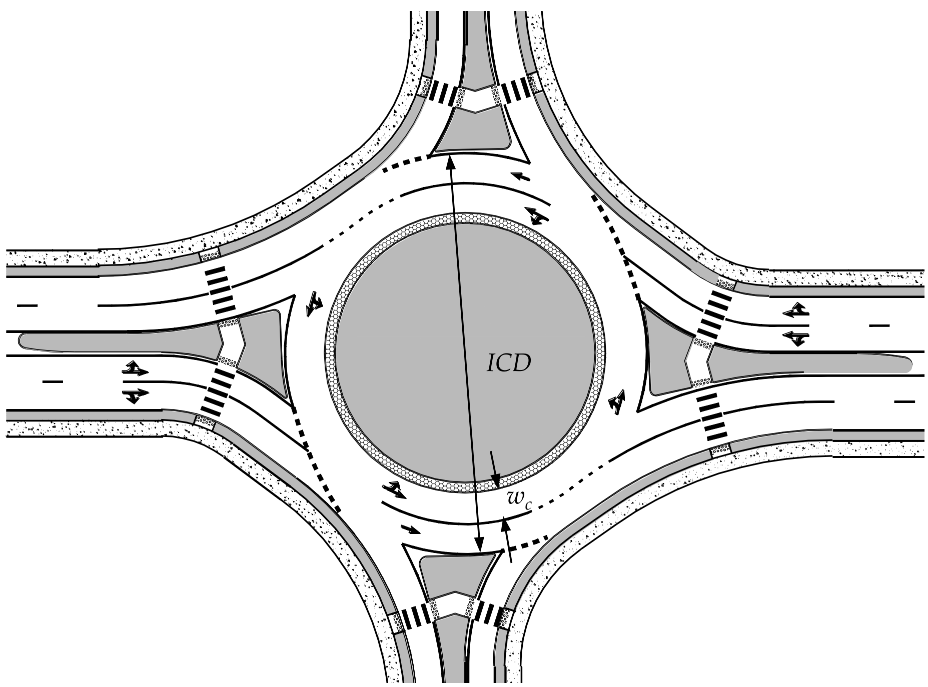
GEOMETRIC DESIGN DATA

This subsection describes the geometric design data listed in Exhibit 30-41. These data describe the additional geometric elements of the roundabouts beyond the geometric elements of the intersections and segments described in Exhibit 18-5.

Inscribed Circle Diameter

The inscribed circle diameter, *ICD*, is the diameter of the largest circle that can be inscribed within the outer edges of the circulatory roadway. The ICD serves as the width of the roundabout. This is illustrated in Exhibit 30-42.

Exhibit 30-42
Illustration of Geometric Design Data



For the purposes of this methodology, if the ICD is variable throughout the roundabout (e.g., to accommodate a variable number of circulating lanes, as illustrated in Exhibit 30-42), the larger dimension should be used.

Number of Circulating Lanes

The number of circulating lanes N_c is the count of circulating lanes immediately downstream of the entry that forms the end of the segment under study.

Average Width of Circulating Lanes

The average width of circulating lanes w_c is measured in the section of circulatory roadway immediately downstream of the entry, that is, the same location where the number of circulating lanes is counted. This is illustrated in Exhibit 30-42.

COMPUTATIONAL STEPS

The computational steps described below are illustrated in the flowchart provided in Exhibit 18-8. The path followed is that of a noncoordinated system with YIELD control.

Step 1: Determine Traffic Demand Adjustments

The models developed for estimating travel speed through a series of roundabouts were calibrated by using roundabouts that were operating below capacity. Neither the capacity estimation procedures for roundabouts in Chapter 22 nor the procedures in this subsection explicitly account for capacity constraint that restricts (or meters) discharge volume from the intersection when the demand volume for an intersection traffic movement exceeds its capacity. Similarly, the methodology does not account for the effect on roundabout operations or travel time that may be created by queue spillback between two roundabouts. The occurrence of any of these conditions should be flagged, and an alternative tool should be considered.

Step 2: Determine Running Time

A procedure for determining running time for a segment bounded by one or more roundabouts is described in this step. It builds on the procedure described in Chapter 18. Each calculation is discussed in the following subparts, which culminate with the calculation of segment running time.

A. Determine Free-Flow Speed

Free-flow speed represents the average running speed of through vehicles traveling along a segment under low-volume conditions and not delayed by traffic control devices or other vehicles. It reflects the effect of the street environment on driver speed choice. Elements of the street environment that influence this choice under free-flow conditions include speed limit, access point density, median type, curb presence, and segment length. Further discussion on free-flow speed can be found in Section 3 of Chapter 18.

Free-flow speed (when the influence of roundabouts at one or both ends of the segment is considered) is calculated by separately determining the free-flow speed influenced by the roundabout at each end of the segment and then comparing these two free-flow speed estimates with the free-flow speed that would be estimated without the presence of roundabouts.

Base Free-Flow Speed

The base free-flow speed is defined to be the free-flow speed on longer segments and is computed the same for segments bounded by roundabouts as for segments bounded by signals. It includes the influence of speed limit, access point density, median type, curb presence, and on-street parking presence. It is computed with Equation 30-72.

Equation 30-72

$$S_{fo} = S_{calib} + S_0 + f_{cs} + f_A + f_{pk}$$

where

S_{fo} = base free-flow speed (mi/h),

S_{calib} = base free-flow speed calibration factor (mi/h),

S_0 = speed constant (mi/h),

f_{CS} = adjustment for cross section (mi/h),

f_A = adjustment for access points (mi/h), and

f_{pk} = adjustment for on-street parking (mi/h).

The speed constant and adjustment factors used in Equation 30-72 are listed in Exhibit 30-43. The exhibit is the same as Exhibit 18-11, except that the width of the signalized intersection used in the calculation for the adjustment for access points f_A has been replaced with the inscribed circle diameter of the roundabout, and the range of speed limits is restricted to the validity range for this method. Equations provided in the table footnote can also be used to compute these adjustment factors for conditions not shown in the exhibit. Further discussion of this equation and adjustment factors can be found in Chapter 18.

Exhibit 30-43
Base Free-Flow Speed Adjustment Factors

Speed Limit (mi/h)	Speed Constant S_0 (mi/h)[a]	Median Type	Percent with Restrictive Median (%)	Adjustment for Cross Section f_{CS} (mi/h)[b] No Curb	Curb
25	37.4	Restrictive	20	0.3	-0.9
30	39.7		40	0.6	-1.4
35	42.1		60	0.9	-1.8
40	44.4		80	1.2	-2.2
45	46.8		100	1.5	-2.7
50	49.1	Nonrestrictive	Not applicable	0.0	-0.5
		No median	Not applicable	0.0	-0.5

Access Density D_a (points/mi)	Adjustment for Access Points f_A by Lanes N_{th} (mi/h)[c] 1 Lane	2 Lanes	3 Lanes	Percent with On-Street Parking (%)	Adjustment for Parking (mi/h)[d]
0	0.0	0.0	0.0	0	0.0
2	-0.2	-0.1	-0.1	20	-0.6
4	-0.3	-0.2	-0.1	40	-1.2
10	-0.8	-0.4	-0.3	60	-1.8
20	-1.6	-0.8	-0.5	80	-2.4
40	-3.1	-1.6	-1.0	100	-3.0
60	-4.7	-2.3	-1.6		

Notes: [a] $S_0 = 25.6 + 0.47 S_{pl}$, where S_{pl} = posted speed limit (mi/h).
[b] $f_{CS} = 1.5\ p_{rm} - 0.47\ p_{curb} - 3.7\ p_{curb}\ p_{rm}$, where p_{rm} = proportion of link length with restrictive median (decimal) and p_{curb} = proportion of segment with curb on the right-hand side (decimal).
[c] $f_A = -0.078\ D_a / N_{th}$ with $D_a = 5{,}280\ (N_{ap,s} + N_{ap,o})/(L - ICD_i)$, where D_a = access point density on segment (points/mi); N_{th} = number of through lanes on the segment in the subject direction of travel (ln); $N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points); $N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points); L = segment length (ft); and ICD_i = inscribed circle diameter of roundabout (ft).
[d] $f_{pk} = -3.0 \times$ proportion of link length with on-street parking available on the right-hand side (decimal).

Equation 30-72 has been calibrated by using data for many urban street segments collectively located throughout the United States, so the default value of 0.0 mi/h for S_{calib} is believed to yield results that are reasonably representative of driver behavior in most urban areas. However, if desired, a locally representative value can be determined from field-measured estimates of the base free-flow speed for several street segments. The local default value can be established for typical street segments or for specific street types. This calibration factor is determined as the one value that provides a statistically based best fit between the prediction from Equation 30-72 and the field-measured estimates. A procedure for estimating the base free-flow speed from field data is described in Section 6.

Roundabout Geometry and Speed Parameters

The computation of free-flow speed, roundabout influence area, and geometric delay requires measurement or estimation of a series of geometric parameters associated with the roundabout at one or both ends of the segment. These computations are performed separately for each roundabout.

The central island diameter is equal to the inscribed circle diameter minus the width of the circulatory roadway on each side of the central island. The circulatory roadway width is equal to the average width of each circulating lane times the number of circulating lanes. These calculations are combined into a single equation as given in Equation 30-73.

Equation 30-73

$$CID = ICD - 2N_c w_c$$

where

CID = central island diameter (ft),

ICD = inscribed circle diameter (ft),

N_c = number of circulating lane(s), and

w_c = average width of circulating lane(s) (ft).

The circulating speed, S_c, can be approximated by assuming that the circulating path occupies the centerline of the circulatory roadway with a radius equal to half the central island diameter plus half the total width of the circulatory roadway. This radius can be computed with Equation 30-74.

Equation 30-74

$$r_{c,th} = \frac{ICD}{2} + \frac{N_c w_c}{2}$$

where

$r_{c,th}$ = average radius of circulating path of through movement (ft),

ICD = inscribed circle diameter (ft),

N_c = number of circulating lane(s), and

w_c = average width of circulating lane(s) (ft).

The speed associated with this radius can be estimated with Equation 30-75 (*12*), which assumes a negative cross slope of the circulatory roadway of –0.02, typical of many roundabouts.

Equation 30-75

$$S_c = 3.4614 r_{c,th}^{\;0.3673}$$

where

S_c = circulating speed (mi/h), and

$r_{c,th}$ = average radius of circulating path of through movement (ft).

For the purposes of calculating free-flow speed, roundabout influence area, and geometric delay, the segment length is divided into two subsegments. Subsegment 1 consists of the portion of the segment from the yield line of the upstream roundabout to the midpoint between the two roundabouts, defined as halfway between the cross-street centerlines of the two roundabouts. Subsegment 2 consists of the portion of the segment from this midpoint to the yield line of the downstream roundabout. The lengths of these subsegments are calculated with Equation 30-76 and Equation 30-77. These dimensions are illustrated in Exhibit 30-44.

Equation 30-76

$$L_1 = \frac{1}{2}\left(L - \frac{ICD_1}{2} + \frac{ICD_2}{2}\right) + \frac{ICD_1}{2}$$

Equation 30-77

$$L_2 = L - L_1$$

where

L_1 = length of Subsegment 1 (ft),

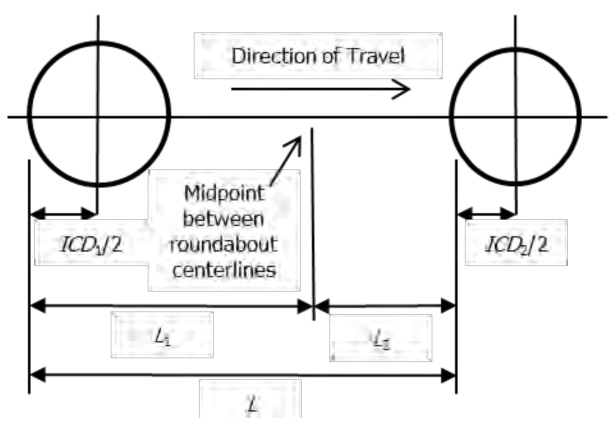
L_2 = length of Subsegment 2 (ft),

L = length of segment (ft),

ICD_1 = inscribed circle diameter of Roundabout 1 (ft), and

ICD_2 = inscribed circle diameter of Roundabout 2 (ft).

Exhibit 30-44
Illustration of Subsegment Dimensions



Free-Flow Speed for Upstream Subsegment (Subsegment 1)

Free-slow speed for Subsegment 1 (the upstream subsegment) is computed in a three-step process by first determining an initial free-flow speed. A roundabout influence area is then computed as the distance over which the geometric features of the roundabout influence travel speed. The initial free-flow speed is then adjusted downward if the roundabout influence area meets or exceeds the length of the subsegment.

The initial free-flow speed for Subsegment 1 is estimated from the subsegment length, posted speed limit, and central island diameter of the roundabout at the upstream end of the segment by using Equation 30-78.

$$S_{f,1,\text{initial}} = 14.6 + 0.0039L_1 + 0.48S_{PL} + 0.02CID_1$$

Equation 30-78

where

$S_{f,1,\text{initial}}$ = initial free-flow speed for Subsegment 1 (mi/h),

L_1 = length of Subsegment 1 (ft),

S_{PL} = posted speed limit (mi/h), and

CID_1 = central island diameter for roundabout at upstream end of Subsegment 1 (ft).

The roundabout influence area for Subsegment 1, RIA_1, is estimated from the free-flow speed and circulating speed with Equation 30-79. This equation yields positive values for inputs within the range limits.

$$RIA_1 = -149.8 + 31.4S_{f,1,\text{initial}} - 22.5S_{c,1}$$

Equation 30-79

where

RIA_1 = roundabout influence area for Subsegment 1 (ft),

$S_{f,1,\text{initial}}$ = initial free-flow speed for Subsegment 1 (mi/h), and

$S_{c,1}$ = through movement circulating speed for roundabout at upstream end of segment (mi/h).

The roundabout influence area is then compared with the length of the subsegment, as shown in Equation 30-80. If the roundabout influence area is equal to or exceeds the length of the subsegment, the subsegment free-flow speed is reduced.

$$S_{f,1} = S_{f,1,\text{initial}} - 4.43 \text{ if } RIA_1 \geq L_1, \text{ else}$$
$$S_{f,1} = S_{f,1,\text{initial}}$$

Equation 30-80

where $S_{f,1}$ is the free-flow speed for Subsegment 1 (mi/h).

Free-Flow Speed for Downstream Subsegment (Subsegment 2)

The initial free-flow speed for Subsegment 2, $S_{f,2,\text{initial}}$, is estimated with Equation 30-81.

$$S_{f,2,\text{initial}} = 15.1 + 0.0037L_2 + 0.43S_{PL} + 0.05CID_2$$

Equation 30-81

where

$S_{f,2,\text{initial}}$ = initial free-flow speed for Subsegment 2 (mi/h),

L_2 = length of Subsegment 2 (ft),

S_{PL} = posted speed limit (mi/h), and

CID_2 = central island diameter for roundabout at downstream end of Subsegment 2 (ft).

The roundabout influence area for the subsegment RIA_2 is estimated from the free-flow speed and downstream circulating speed with Equation 30-82.

Equation 30-82

$$RIA_2 = 165.9 + 13.8S_{f,2,\text{initial}} - 21.1S_{c,2}$$

where

RIA_2 = roundabout influence area for Subsegment 2 (ft),

$S_{f,2,\text{initial}}$ = initial free-flow speed for Subsegment 2 (mi/h), and

$S_{c,2}$ = through movement circulating speed for roundabout at downstream end of subsegment (mi/h).

The roundabout influence area is then compared with the length of the subsegment, as shown in Equation 30-83. If the roundabout influence area is equal to or exceeds the length of the subsegment, the subsegment free-flow speed is reduced to account for the overlap.

Equation 30-83

$$S_{f,2} = S_{f,2,\text{initial}} - 4.73 \text{ if } RIA_2 \geq L_2\text{, else}$$
$$S_{f,2} = S_{f,2,\text{initial}}$$

where $S_{f,2}$ is the free-flow speed for Subsegment 2 (mi/h).

Free-Flow Speed Without Influence of Roundabouts

The calculation for free-flow speed without the geometric influence of roundabouts is the same as for segments bounded by signalized intersections, as provided in Chapter 18. Equation 30-84 is used to compute the value of an adjustment factor that accounts for the influence of short spacing of boundary intersections.

Equation 30-84

$$f_L = 1.02 - 4.7\frac{S_{fo} - 19.5}{\max(L_s, 400)} \leq 1.0$$

where

f_L = boundary intersection spacing adjustment factor;

S_{fo} = base free-flow speed (mi/h); and

L_s = distance between adjacent boundary intersections that (*a*) bracket the subject segment and (*b*) each have a type of control that can impose on the subject through movement a legal requirement to stop or yield, such as a roundabout (ft).

The predicted free-flow speed without the geometric influence of roundabouts is computed with Equation 30-85 on the basis of estimates of base free-flow speed and the signal spacing adjustment factor.

Equation 30-85

$$S_{f,non\text{-}rbt} = S_{fo}\, f_L \geq S_{pl}$$

where $S_{f,non\text{-}rbt}$ is the free-flow speed for nonroundabout segments (mi/h) and S_{pl} is the posted speed limit. If the speed obtained from Equation 30-85 is less than the speed limit, the speed limit is used.

Free-Flow Speed

The free-flow speeds for each subsegment are then compared with each other and with the nonroundabout free-flow speed with Equation 30-86. The lowest of these speeds is the governing free-flow speed for the segment. The analyst is cautioned that if the result of this calculation is outside the validity range presented in Exhibit 30-40, the calculation is an extrapolation of the model. Note that the resulting free-flow speed for a segment bounded by one or more roundabouts may be lower than the posted speed, even though the nonroundabout free-flow speed is constrained by the posted speed in accordance with the motorized vehicle methodology in Chapter 18.

$$S_f = \min\left(S_{f,1}, S_{f,2}, S_{f,non\text{-}rbt}\right)$$

Equation 30-86

B. Compute Adjustment for Vehicle Proximity

This step is the same as in Chapter 18.

C. Compute Delay due to Turning Vehicles

This step is the same as in Chapter 18.

D. Estimate Delay due to Other Sources

This step is the same as in Chapter 18.

E. Compute Segment Running Time

Equation 30-87 is used to compute the segment running time, which is based on Equation 18-7. It incorporates the conditions specified in Chapter 18 for a yield-controlled boundary exiting the segment: a start-up lost time of 2.5 s and the influence of the volume-to-capacity ratio of the roundabout entry.

$$t_R = \frac{3.5}{0.0025\,L} \times \min\left(\frac{v_{th}}{c_{th}}, 1.00\right) + \frac{3{,}600\,L}{5{,}280\,S_f} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{\text{other}}$$

Equation 30-87

where

t_R = segment running time (s),

L = segment length (ft),

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

c_{th} = through-movement capacity (veh/h),

f_v = proximity adjustment factor,

$d_{ap,i}$ = delay due to left and right turns from the street into access point intersection i (s/veh),

N_{ap} = number of influential access point approaches along the segment = $N_{ap,s} + p_{ap,lt} N_{ap,o}$ (points),

$N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points),

$N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points),

$p_{ap,lt}$ = proportion of $N_{ap,o}$ that can be accessed by a left turn from the subject direction of travel, and

d_{other} = delay due to other sources along the segment (e.g., curb parking or pedestrians) (s/veh).

The variables v_{th} and c_{th} used in Equation 30-87 apply to the through movement exiting the segment at the boundary roundabout.

Step 3: Determine the Proportion Arriving During Green

This step does not apply to a segment with a downstream roundabout. The methodology does not account for the possible effects of an upstream signal on a downstream roundabout.

Step 4: Determine Signal Phase Duration

This step does not apply to a segment with a downstream roundabout.

Step 5: Determine Through Delay

The through delay for a segment with a roundabout at one or both ends is computed as a combination of control delay and geometric delay.

The procedure for computing the control delay at a roundabout at the downstream end of a segment is provided in Chapter 22, which determines the control delay for a roundabout on a lane-by-lane basis. For an approach with one lane, the through control delay is equal to the control delay of the lane. For an approach with two lanes, the through control delay is computed by allocating the control delay in each lane in proportion to the through traffic in each lane by using Equation 30-88.

Equation 30-88

$$d_{\text{control},t} = \frac{d_{LL}\ v_{LL} P_{LL,T} + d_{RL}\ v_{RL} P_{RL,T}}{v_{th}}$$

where

$d_{\text{control},t}$ = through control delay (s/veh),

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

d_{LL} = control delay in left lane (s/veh),

v_{LL} = demand flow rate in left lane (veh/h),

d_{RL} = control delay in right lane (s/veh),

v_{RL} = demand flow rate in right lane (veh/h),

$P_{LL,T}$ = proportion of through-movement vehicles in the left lane (decimal), and

$P_{RL,T}$ = proportion of through-movement vehicles in the right lane (decimal).

Geometric delay is calculated separately for the presence of a roundabout on the two subsegments. If a roundabout is present on the upstream end of Subsegment 1 (regardless of the control present at the downstream end of Subsegment 2), the geometric delay for the upstream portion of the segment $d_{geom,1}$ is calculated with Equation 30-89. If the upstream end of the segment is controlled by a signalized or stop-controlled intersection or is uncontrolled, $d_{geom,1} = 0$.

Equation 30-89

$$d_{geom,1} = \max\left[-2.63 + 0.09 S_f + 0.625 ICD_1 \left(\frac{1}{S_{c,1}} - \frac{1}{S_f}\right), 0\right]$$

where $d_{geom,1}$ is the geometric delay for Subsegment 1 (s/veh).

If a roundabout is present on the downstream end of the segment (regardless of the control present at the upstream end), the geometric delay for the downstream portion of the segment $d_{geom,2}$ is calculated with Equation 30-90. If the upstream end of the segment is controlled by a signalized or stop-controlled intersection or is uncontrolled, $d_{geom,2} = 0$.

$$d_{geom,2} = \max(1.57 + 0.11S_f - 0.21S_{c,2}, 0)$$

Equation 30-90

where $d_{geom,2}$ is the geometric delay for Subsegment 2 (s/veh).

The analyst is cautioned that if these calculations result in one or more geometric delay estimates outside the validity range presented in Exhibit 30-40, the calculation is an extrapolation of the model.

The through delay d_t is computed as the sum of control and geometric delays, as given in Equation 30-91.

$$d_t = d_{\text{control},t} + d_{geom,1} + d_{geom,2}$$

Equation 30-91

Step 6: Determine Through Stop Rate

As noted in Chapter 18, the stop rate at a YIELD-controlled approach will vary with conflicting demand. It can be estimated (in stops per vehicle) as equal to the volume-to-capacity ratio of the through movement at the boundary intersection. This approach recognizes that YIELD control does not require drivers to come to a complete stop when there is no conflicting traffic. The through stop rate h is computed as given in Equation 30-92. The methodology does not apply for volume-to-capacity ratios exceeding 1.0.

$$h = \min\left(\frac{v_{th}}{c_{th}}, 1.00\right)$$

Equation 30-92

Step 7: Determine Travel Speed

This step is the same as for Chapter 18.

Step 8: Determine Spatial Stop Rate

This step is the same as for Chapter 18.

Step 9: Determine LOS

This step is the same as for Chapter 18. The base free-flow speed for the estimation of LOS is the same base free-flow speed as determined in Chapter 18.

Step 10: Determine Motor Vehicle Traveler Perception Score

Research has not been conducted on the traveler's perception of service quality for roundabouts in a manner that can be integrated into this methodology. As a result, the motor vehicle traveler perception score for a segment bounded by a roundabout is undefined and this step is not applicable for the evaluation of roundabout segments.

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Enhancing Pedestrian Volume Estimation and Developing HCM Pedestrian Methodologies for Safe and Sustainable Communities

Chapter 31: Signalized Intersections: Supplemental

Version 6.1 Final Draft

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This chapter contains new Example Problems 4 and 5 and revisions to Example Problem 2 to incorporate changes and additions to the pedestrian methodologies in Chapter 19. A small change in the introduction, Section 1, has also been made to reflect the changes to the example problems. These revisions were developed through NCHRP Project 17-87.

CHAPTER 31
SIGNALIZED INTERSECTIONS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 31 is the supplemental chapter for Chapter 19, Signalized Intersections, which is found in Volume 3 of the *Highway Capacity Manual* (HCM). This chapter presents detailed information about the following aspects of the Chapter 19 motorized vehicle methodology:

- Procedures are described for computing actuated phase duration and pretimed phase duration.
- Procedures are described for computing saturation flow rate adjustment factors to account for the presence of pedestrians, bicycles, and work zones.
- A procedure is described for computing uniform delay by using the queue accumulation polygon (QAP) concept. The procedure is extended to shared-lane lane groups and lane groups with permitted turn movements.
- A procedure is described for computing queue length and queue storage ratio.

This chapter provides a simplified version of the Chapter 19 motorized vehicle methodology that is suitable for planning applications. The chapter also describes techniques for measuring control delay and saturation flow rate in the field and provides details about the computational engine that implements the Chapter 19 motorized vehicle methodology. Finally, this chapter provides five example problems that demonstrate the application of the motorized vehicle, pedestrian, and bicycle methodologies to a signalized intersection.

2. CAPACITY AND PHASE DURATION

This section describes five procedures related to the calculation of capacity and phase duration. The first procedure is used to calculate the average duration of an actuated phase, and the second is used to calculate the lane volume distribution on multilane intersection approaches. The third procedure focuses on the calculation of phase duration for pretimed intersection operation. The fourth procedure is used to compute the pedestrian and bicycle saturation flow rate adjustment factors, and the fifth computes the work zone saturation flow rate adjustment factor. Each procedure is described in a separate subsection.

ACTUATED PHASE DURATION

This subsection describes a procedure for estimating the average phase duration for an intersection that is operating with actuated control. When appropriate, the description is extended to include techniques for estimating the duration of noncoordinated and coordinated phases. Unless stated otherwise, a noncoordinated phase is modeled as an actuated phase in this methodology.

This subsection consists of the following seven parts:

- Concepts,
- Volume computations,
- Queue accumulation polygon,
- Maximum allowable headway,
- Equivalent maximum green,
- Average phase duration, and
- Probability of max-out.

The last six parts in the list above describe a series of calculations that are completed in the sequence shown to obtain estimates of average phase duration and the probability of phase termination by extension to its maximum green limit (i.e., max-out).

Concepts

The duration of an actuated phase is composed of five time periods, as shown in Equation 31-1. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the effective green time associated with queue clearance. 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 a max-out. The fourth period represents the yellow change interval, and the last period represents the red clearance interval.

Equation 31-1

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

where

D_p = phase duration (s),

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

Y = yellow change interval (s),

R_c = red clearance interval (s),

g_s = queue service time (s),

g_e = green extension time (s).

The relationship between the variables in Equation 31-1 is shown in Exhibit 31-1 with a QAP. Key variables shown in the exhibit are defined for Equation 31-1 and in the following list:

q_r = arrival flow rate during the effective red time = $(1 - P)\, q\, C/r$ (veh/s),

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

r = effective red time = $C - g$ (s),

g = effective green time (s),

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

q_g = arrival flow rate during the effective green time = $P\, q\, C/g$ (veh/s),

q = arrival flow rate (veh/s),

Q_r = queue size at the end of the effective red time = $q_r\, r$ (veh),

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

e = extension of effective green = 2.0 (s).

Exhibit 31-1
Time Elements Influencing
Actuated Phase Duration

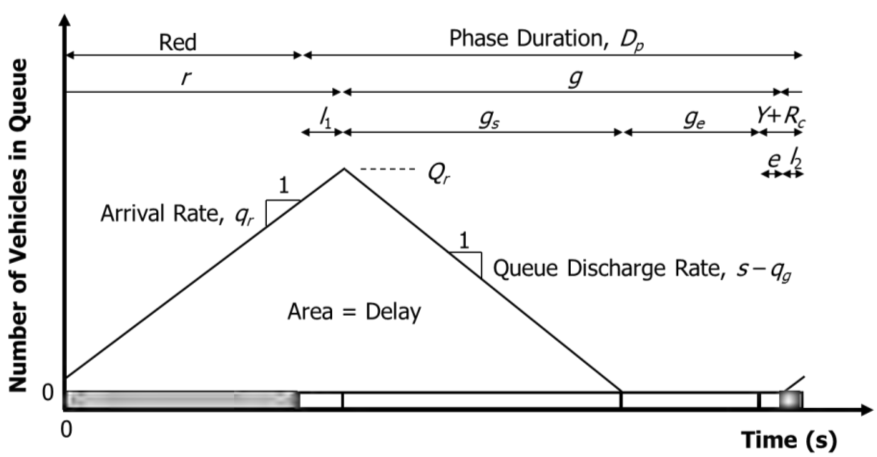


Exhibit 31-1 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the green interval starts. At this time, the queue begins to discharge at a rate equal to the saturation flow rate s less the arrival rate during green q_g. The queue clears g_s seconds after it first begins to discharge. Thereafter, random vehicle arrivals are detected and cause the green interval to be extended. Eventually, a gap occurs in traffic (or the maximum green limit is reached), and the green interval ends. The end of the green interval coincides with the end of the extension time g_e.

The effective green time for the phase is computed with Equation 31-2.

Equation 31-2

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

where all variables are as previously defined.

Coordinated Phase Duration

The duration of a coordinated phase is dictated by the cycle length and the force-off settings for the noncoordinated phases. These settings define the points in the signal cycle at which each noncoordinated phase must end. The force-off settings are used to ensure the coordinated phases receive a green indication at a specific time in the cycle. Presumably, this time is synchronized with the coordinated phase time at the adjacent intersections so that traffic progresses along the street segment. In general, the duration of a coordinated phase is equal to the cycle length less the time allocated to the conflicting phase in the same ring and less the time allocated to the minor-street phases. Detectors are not typically assigned to the coordinated phase, and this phase is not typically extended by the vehicles it serves.

Noncoordinated Phase Duration

The duration of a noncoordinated phase is dictated by traffic demand in much the same manner as for an actuated phase. However, the noncoordinated phase duration is typically constrained by its force-off setting (rather than a maximum green setting). A noncoordinated phase is referred to here and modeled as an *actuated* phase.

Right-Turn Overlap Duration

If a right-turn lane group is operated in a protected or protected-permitted mode, then the protected indication is assumed to be provided as a right-turn overlap with the complementary left-turn phase on the intersecting roadway. In this manner, the right-turn protected interval duration is dictated by the duration of the complementary left-turn phase (which is determined by the left-turn phase settings, left-turn detection, and left-turn volume). The procedures described in this subsection are used to determine the average duration of the complementary left-turn lane phase (and thus the protected right-turn interval duration).

The right-turn permitted interval duration is dictated by the phase settings, detection, and volume associated with the right-turn movement and its adjacent through movement. The procedures described in this subsection are used to determine the average duration of the phase serving the right-turn movement in a permitted manner.

Volume Computations

This subsection describes the calculations needed to quantify the time rate of calls submitted to the controller by the detectors. Two call rates are computed for each signal phase. The first rate represents the flow rate of calls for green extension that arrive during the green interval. The second call rate represents the flow rate of calls for phase activation that arrive during the red indication.

A. Call Rate to Extend Green

The call rate to extend the green indication for a given phase is based on the flow rate of the lane groups served by the phase. The call rate is represented in the analysis by the flow rate parameter. This parameter represents an adjusted flow rate that accounts for the tendency of drivers to form "bunches" (i.e., randomly formed platoons). The flow rate parameter for the phase is computed as shown by Equation 31-3 with Equation 31-4 and Equation 31-5.

$$\lambda^* = \sum_{i=1}^{m} \lambda_i$$

Equation 31-3

with

$$\lambda_i = \frac{\varphi_i\, q_i}{1 - \Delta_i\, q_i}$$

Equation 31-4

$$\varphi_i = e^{-b_i\, \Delta_i\, q_i}$$

Equation 31-5

where

- λ^* = flow rate parameter for the phase (veh/s);
- λ_i = flow rate parameter for lane group i (i = 1, 2, . . . , m) (veh/s);
- ϕ_i = proportion of free (unbunched) vehicles in lane group i (decimal);
- q_i = arrival flow rate for lane group $i = v_i/3{,}600$ (veh/s);
- v_i = demand flow rate for lane group i (veh/h);
- Δ_i = headway of bunched vehicle stream in lane group i; = 1.5 s for single-lane lane group, 0.5 s otherwise (s/veh);
- m = number of lane groups served during the phase; and
- b_i = bunching factor for lane group i (0.6, 0.5, and 0.8 for lane groups with 1, 2, and 3 or more lanes, respectively).

Using Equation 31-6, Equation 31-7, and Equation 31-8, it is also useful to compute the following three variables for each phase. These variables are used in a later step to compute green extension time.

$$\varphi^* = e^{-\sum_{i=1}^{m} b_i\, \Delta_i\, q_i}$$

Equation 31-6

$$\Delta^* = \frac{\sum_{i=1}^{m} \lambda_i\, \Delta_i}{\lambda^*}$$

Equation 31-7

$$q^* = \sum_{i=1}^{m} q_i$$

Equation 31-8

where

- ϕ^* = combined proportion of free (unbunched) vehicles for the phase (decimal),
- Δ^* = equivalent headway of bunched vehicle stream served by the phase (s/veh), and
- q^* = arrival flow rate for the phase (veh/s), and

all other variables are as previously defined.

The call rate for green extension for a phase that does not end at a barrier is equal to the flow rate parameter λ^*. If two phases terminate at a common barrier (i.e., one phase in each ring) and simultaneous gap-out is enabled, then the call rate for either phase is based on the combined set of lane groups being served by the two phases. To model this behavior, the lane group parameters for each phase are combined to estimate the call rate for green extension. Specifically, the variable m in the preceding six equations is modified to represent the combined number of lane groups served by both phases.

The following rules are evaluated to determine the number of lane groups served m if simultaneous gap-out is enabled. They are described for the case in which Phases 2, 6, 4, and 8 end at the barrier (as shown in Exhibit 19-2). The rules should be modified if other phase pairs end at the barrier.

1. If Phases 2 and 6 have simultaneous gap-out enabled, then the lane groups associated with Phase 2 are combined with the lane groups associated with Phase 6 in applying Equation 31-3 through Equation 31-8 for Phase 6. Similarly, the lane groups associated with Phase 6 are combined with the lane groups associated with Phase 2 in applying these equations for Phase 2.
2. If Phases 4 and 8 have simultaneous gap-out enabled, then the lane groups associated with Phase 4 are combined with the lane groups associated with Phase 8 in evaluating Phase 8. Similarly, the lane groups associated with Phase 8 are combined with the lane groups associated with Phase 4 in evaluating Phase 4.

B. Call Rate to Activate a Phase

The call rate to activate a phase is used to determine the probability that the phase is activated in the forthcoming cycle sequence. This rate is based on the arrival flow rate of the traffic movements served by the phase and whether the phase is associated with dual entry. Vehicles or pedestrians can call a phase, so a separate call rate is computed for each traffic movement.

i. Determine Phase Vehicular Flow Rate. The vehicular flow rate associated with a phase depends on the type of movements it serves as well as the approach lane allocation. The following rules apply in determining the phase vehicular flow rate:

1. If the phase exclusively serves a left-turn movement, then the phase vehicular flow rate is equal to the left-turn movement flow rate.
2. If the phase serves a through or right-turn movement and there is no exclusive left-turn phase for the adjacent left-turn movement, then the phase vehicular flow rate equals the approach flow rate.
3. If the phase serves a through or right-turn movement and there is an exclusive left-turn phase for the adjacent left-turn movement, then
    a. If there is a left-turn bay, then the phase vehicular flow rate equals the sum of the through and right-turn movement flow rates.
    b. If there is no left-turn bay, then the phase vehicular flow rate equals the approach flow rate.

c. If split phasing is used, then the phase vehicular flow rate equals the approach flow rate.

ii. Determine Activating Vehicular Call Rate. The activating vehicular call rate q_v^* is equal to the phase vehicular flow rate divided by 3,600 to convert it to units of vehicles per second. If dual entry is activated for a phase, then the activation call rate must be modified by adding its original rate to that of both concurrent phases. For example, if Phase 2 is set for dual entry, then the modified Phase 2 activation call rate equals the original Phase 2 activation call rate plus the activation rate of Phase 5 and the activation rate of Phase 6. In this manner, Phase 2 is activated when demand is present for Phase 2, 5, or 6.

iii. Determine Activating Pedestrian Call Rate. The activating pedestrian call rate q_p^* is equal to the pedestrian flow rate associated with the subject approach divided by 3,600 to convert it to units of pedestrians per second. If dual entry is activated for a phase, then the activation call rate must be modified by adding its original rate to that of the opposing through phase. For example, if Phase 2 is set for dual entry, then the modified Phase 2 activation call rate equals the original Phase 2 activation call rate plus the activation rate of Phase 6. In this manner, Phase 2 is activated when pedestrian demand is present for Phase 2 or 6.

Queue Accumulation Polygon

This subsection summarizes the procedure used to construct the QAP associated with a lane group. This polygon defines the queue size for a traffic movement as a function of time during the cycle. The procedure is described more fully in Section 3; it is discussed here to illustrate its use in calculating queue service time.

For polygon construction, all flow rate variables are converted to common units of vehicles per second per lane. The presentation in this subsection is based on these units for q and s. If the flow rate q exceeds the lane capacity, then it is set to equal this capacity.

A polygon is shown in Exhibit 31-1 for a through movement in an exclusive lane. At the start of the effective red, vehicles arrive at a rate of q_r and accumulate to a length of Q_r vehicles at the time the effective green begins. Thereafter, the queue begins to discharge at a rate of $s - q_g$ until it clears after g_s seconds. The queue service time g_s represents the time required to serve the queue present at the end of effective red Q_r plus any additional arrivals that join the queue before it fully clears. Queue service time is computed as $Q_r/(s - q_g)$. Substituting the variable relationships in the previous variable list into this equation yields Equation 31-9 for estimating queue service time.

$$g_s = \frac{q\ C\ (1 - P)}{\frac{s}{3{,}600} - q\ C\ (P/g)}$$

Equation 31-9

where P is the proportion of vehicles arriving during the green indication (decimal), s is the adjusted saturation flow rate (veh/h/ln), and all other variables are as previously defined.

The polygon in Exhibit 31-1 applies to some types of lane groups. Other polygon shapes are possible. A detailed procedure for constructing polygons is described in Section 3.

Maximum Allowable Headway

This subsection describes a procedure for calculating the maximum allowable headway (MAH) for the detection associated with a phase. It consists of two steps. Step A computes MAH for each lane group served by the subject phase. Step B combines MAH into an equivalent MAH for the phase. The latter step is used when a phase serves two or more lane groups or when simultaneous gap-out is enabled.

The procedure addresses the situation in which there is one zone of detection per lane. This type of detection is referred to here as *stop-line detection* because the detection zone is typically located at the stop line. However, some agencies prefer to locate the detection zone at a specified distance upstream from the stop line. This procedure can be used to evaluate any single-detector-per-lane design, provided the detector is located so that only the subject traffic movement travels over this detector during normal operation.

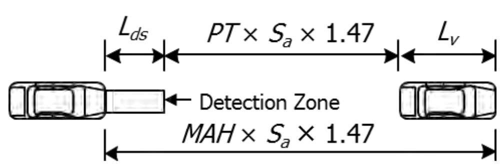
The detector length and detection mode input data are specified by movement group. When these data describe a through movement group, it is reasonable to assume they also describe the detection in any shared-lane lane groups that serve the through movement. This assumption allows the movement group inputs to describe the associated lane group values, and the analysis can proceed on a lane-group basis. However, if this assumption is not valid or if information about the detection design for each lane is known, then the procedure can be extended to the calculation of MAH for each lane. The lane-specific MAHs would then be combined for the phase that serves these lanes.

Concepts

MAH represents the maximum time that can elapse between successive calls for service without terminating the phase by gap-out. It is useful for describing the detection design and signal settings associated with a phase. MAH depends on the number of detectors serving the lane group, the length of these detectors, and the average vehicle speed in the lane group.

The relationship between passage time PT, detection zone length L_{ds}, vehicle length L_v, average speed S_a, and MAH is shown in Exhibit 31-2. The two vehicles shown are traveling from left to right and have a headway equal to MAH so that the second vehicle arrives at the detector the instant the passage time is set to time out.

Exhibit 31-2
Detection Design and
Maximum Allowable Headway



According to Exhibit 31-2, Equation 31-10 with Equation 31-11 can be derived for estimating MAH for stop-line detection operating in the presence mode.

$$MAH = PT + \frac{L_{ds} + L_v}{1.47\, S_a}$$

Equation 31-10

with

$$L_v = L_{pc}(1 - 0.01\, P_{HV}) + 0.01\, L_{HV} P_{HV} - D_{sv}$$

Equation 31-11

where

MAH = maximum allowable headway (s/veh),

PT = passage time setting (s),

L_{ds} = length of the stop-line detection zone (ft),

L_v = detected length of the vehicle (ft),

S_a = average speed on the intersection approach (mi/h),

L_{pc} = stored passenger car lane length = 25 (ft),

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

L_{HV} = stored heavy-vehicle lane length = 45 (ft), and

D_{sv} = distance between stored vehicles = 8 (ft).

The average speed on the intersection approach can be estimated with Equation 31-12.

$$S_a = 0.90\,(25.6 + 0.47\, S_{pl})$$

Equation 31-12

where S_{pl} is the posted speed limit (mi/h).

Equation 31-10 is derived for the typical case in which the detection unit is operating in the presence mode. If it is operating in the pulse mode, then MAH equals the passage time setting PT.

A. Determine Maximum Allowable Headway

Equation 31-10 has been modified to adapt it to various combinations of lane use and left-turn operation. A family of equations is presented in this step. The appropriate equation is selected for the subject lane group and then used to compute the corresponding MAH.

The equations presented in this step are derived for the typical case in which the detection unit is operating in the presence mode. If a detector is operating in the pulse mode, then MAH equals the passage time setting PT.

MAH for lane groups serving through vehicles is calculated with Equation 31-13.

$$MAH_{th} = PT_{th} + \frac{L_{ds,th} + L_v}{1.47\, S_a}$$

Equation 31-13

where

MAH_{th} = maximum allowable headway for through vehicles (s/veh),

PT_{th} = passage time setting for phase serving through vehicles (s),

$L_{ds,th}$ = length of the stop-line detection zone in the through lanes (ft), and

S_a = average speed on the intersection approach (mi/h).

MAH for a left-turn movement served in exclusive lanes with the protected mode (or protected-permitted mode) is based on Equation 31-13, but the equation is adjusted as shown in Equation 31-14 to account for the slower speed of the left-turn movement.

Equation 31-14

$$MAH_{lt,e,p} = PT_{lt} + \frac{L_{ds,lt} + L_v}{1.47\ S_a} + \frac{E_L - 1}{s_o/3{,}600}$$

where

$MAH_{lt,e,p}$ = maximum allowable headway for protected left-turning vehicles in exclusive lane (s/veh),

PT_{lt} = passage time setting for phase serving the left-turning vehicles (s),

$L_{ds,lt}$ = length of the stop-line detection zone in the left-turn lanes (ft),

E_L = equivalent number of through cars for a protected left-turning vehicle = 1.05, and

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

MAH for left-turning vehicles served in a shared lane with the protected-permitted mode is calculated as shown in Equation 31-15.

Equation 31-15

$$MAH_{lt,s,p} = MAH_{th} + \frac{E_L - 1}{s_o/3{,}600}$$

where $MAH_{lt,s,p}$ is the maximum allowable headway for protected left-turning vehicles in a shared lane (s/veh).

MAH for left-turning vehicles served in an exclusive lane with the permitted mode is adjusted to account for the longer headway of the turning vehicle. In this case, the longer headway includes the time spent waiting for an acceptable gap in the opposing traffic stream. Equation 31-16 addresses these adjustments.

Equation 31-16

$$MAH_{lt,e} = PT_{th} + \frac{L_{ds,lt} + L_v}{1.47\ S_a} + \frac{3{,}600}{s_l} - t_{fh}$$

where

$MAH_{lt,e}$ = maximum allowable headway for permitted left-turning vehicles in exclusive lane (s/veh),

s_l = saturation flow rate in exclusive left-turn lane group with permitted operation (veh/h/ln), and

t_{fh} = follow-up headway = 2.5 (s).

MAH for right-turning vehicles served in an exclusive lane with the protected mode is computed with Equation 31-17.

Equation 31-17

$$MAH_{rt,e,p} = PT_{rt} + \frac{L_{ds,rt} + L_v}{1.47\ S_a} + \frac{E_R - 1}{s_o/3{,}600}$$

where

$MAH_{rt,e,p}$ = maximum allowable headway for protected right-turning vehicles in exclusive lane (s/veh),

PT_{rt} = passage time setting for phase serving right-turning vehicles (s),

E_R = equivalent number of through cars for a protected right-turning vehicle = 1.18, and

$L_{ds,rt}$ = length of the stop-line detection zone in the right-turn lanes (ft).

If the variable E_R in Equation 31-17 is divided by the pedestrian–bicycle saturation flow rate adjustment factor f_{Rpb} and PT_{th} is substituted for PT_{rt}, then the equation can be used to estimate $MAH_{rt,e}$ for permitted right-turning vehicles in an exclusive lane.

Equation 31-18 and Equation 31-19, respectively, are used to estimate MAH for left- and right-turning vehicles that are served in a shared lane with the permitted mode.

$$MAH_{lt,s} = MAH_{th} + \frac{3{,}600}{s_l} - t_{fh}$$

Equation 31-18

$$MAH_{rt,s} = MAH_{th} + \frac{\left(E_R/f_{Rpb}\right) - 1}{s_o/3{,}600}$$

Equation 31-19

where $MAH_{lt,s}$ is the maximum allowable headway for permitted left-turning vehicles in a shared lane (s/veh), and $MAH_{rt,s}$ is the maximum allowable headway for permitted right-turning vehicles in a shared lane (s/veh).

B. Determine Equivalent Maximum Allowable Headway

The equivalent MAH (i.e., *MAH**) is calculated for cases in which more than one lane group is served by a phase. It is also calculated for phases that end at a barrier and that are specified in the controller as needing to gap out at the same time as a phase in the other ring. The following rules are used to compute the equivalent MAH:

1. If simultaneous gap-out is not enabled, or the phase does not end at the barrier, then
   a. If the phase serves only one movement, then *MAH** for the phase equals the MAH computed for the corresponding lane group.
   b. This rule subset applies when the phase serves all movements and there is no exclusive left-turn phase for the approach (i.e., it operates with the permitted mode). The equations shown apply to the most general case in which a left-turn, through, and right-turn movement exist and a through lane group exists. If any of these movements or lane groups do not exist, then their corresponding flow rate parameter equals 0.0 veh/s.
      i. If there is no left-turn lane group or right-turn lane group (i.e., shared lanes), then *MAH** for the phase is computed from Equation 31-20.

Equation 31-20

$$MAH^* = \frac{P_L \lambda_{sl} MAH_{lt,s} + [(1 - P_L)\lambda_{sl} + \lambda_t + (1 - P_R)\lambda_{sr}]MAH_{th} + P_R \lambda_{sr} MAH_{rt,s}}{\lambda_{sl} + \lambda_t + \lambda_{sr}}$$

where

λ_{sl} = flow rate parameter for shared left-turn and through lane group (veh/s),

λ_t = flow rate parameter for exclusive through lane group (veh/s),

λ_{sr} = flow rate parameter for shared right-turn and through lane group (veh/s),

P_L = proportion of left-turning vehicles in the shared lane (decimal), and

P_R = proportion of right-turning vehicles in the shared lane (decimal).

ii. If there is a right-turn lane group but no left-turn lane group, then Equation 31-21 is applicable.

Equation 31-21

$$MAH^* = \frac{P_L\, \lambda_{sl}\, MAH_{lt,s} + [(1 - P_L)\, \lambda_{sl} + \lambda_t]\, MAH_{th} + \lambda_r\, MAH_{rt,e}}{\lambda_{sl} + \lambda_t + \lambda_r}$$

where λ_r is the flow rate parameter for the exclusive right-turn lane group (veh/s).

iii. If there is a left-turn lane group but no right-turn lane group, then *MAH** for the phase is computed with Equation 31-22.

Equation 31-22

$$MAH^* = \frac{\lambda_l\, MAH_{lt,e} + [\lambda_t + (1 - P_R)\, \lambda_{sr}]\, MAH_{th} + P_R\, \lambda_{sr}\, MAH_{rt,s}}{\lambda_l + \lambda_t + \lambda_{sr}}$$

where λ_l is the flow rate parameter for the exclusive left-turn lane group (veh/s).

iv. If there is a left-turn lane group and a right-turn lane group, then *MAH** for the phase is computed with Equation 31-23.

Equation 31-23

$$MAH^* = \frac{\lambda_l\, MAH_{lt,e} + \lambda_t\, MAH_{th} + \lambda_r\, MAH_{rt,e}}{\lambda_l + \lambda_t + \lambda_r}$$

c. If the phase serves only a through lane group, right-turn lane group, or both, then

i. If there is a right-turn lane group and a through lane group, then *MAH** for the phase is computed with Equation 31-24.

Equation 31-24

$$MAH^* = \frac{\lambda_t\, MAH_{th} + \lambda_r\, MAH_{rt,e}}{\lambda_t + \lambda_r}$$

ii. If there is a shared right-turn and through lane group, then *MAH** for the phase is computed with Equation 31-25.

Equation 31-25

$$MAH^* = \frac{[\lambda_t + (1 - P_R)\, \lambda_{sr}]\, MAH_{th} + P_R\, \lambda_{sr}\, MAH_{rt,s}}{\lambda_t + \lambda_{sr}}$$

d. If the phase serves all approach movements using split phasing, then

i. If there is one lane group (i.e., a shared lane), then *MAH** for the phase equals the MAH computed for the lane group.

ii. If there is more than one lane group, then MAH^* is computed with the equations in previous Rule 1.b, but $MAH_{lt,e,p}$ is substituted for $MAH_{lt,e}$, and $MAH_{lt,s,p}$ is substituted for $MAH_{lt,s}$.

e. If the phase has protected-permitted operation with a shared left-turn and through lane, then the equations in previous Rule 1.b (i.e., 1.b.i and 1.b.ii) apply. The detection for this operation does not influence the duration of the left-turn phase. The left-turn phase will be set to minimum recall and will extend to its minimum value before terminating.

2. If simultaneous gap-out is enabled and the phase ends at the barrier, then MAH^* for the phase is computed with Equation 31-26, where the summations shown are for all lane groups served by the subject (or concurrent) phase.

$$MAH^* = \frac{MAH \sum \lambda_i + MAH_c \sum \lambda_{c,i}}{\sum \lambda_i + \sum \lambda_{c,i}}$$

Equation 31-26

where

MAH^* = equivalent maximum allowable headway for the phase (s/veh),

MAH_c = maximum allowable headway for the concurrent phase that also ends at the barrier (s/veh), and

$\lambda_{c,i}$ = flow rate parameter for lane group i served in the concurrent phase that also ends at the barrier (veh/s).

When there is split phasing, there are no concurrent phases, and Equation 31-26 does not apply.

Equivalent Maximum Green

In coordinated-actuated operation, the force-off points are used to constrain the duration of the noncoordinated phases. Although the maximum green setting is also available to provide additional constraint, it is not commonly used. In fact, the default mode in most modern controllers is to inhibit the maximum green timer when the controller is used in a coordinated signal system.

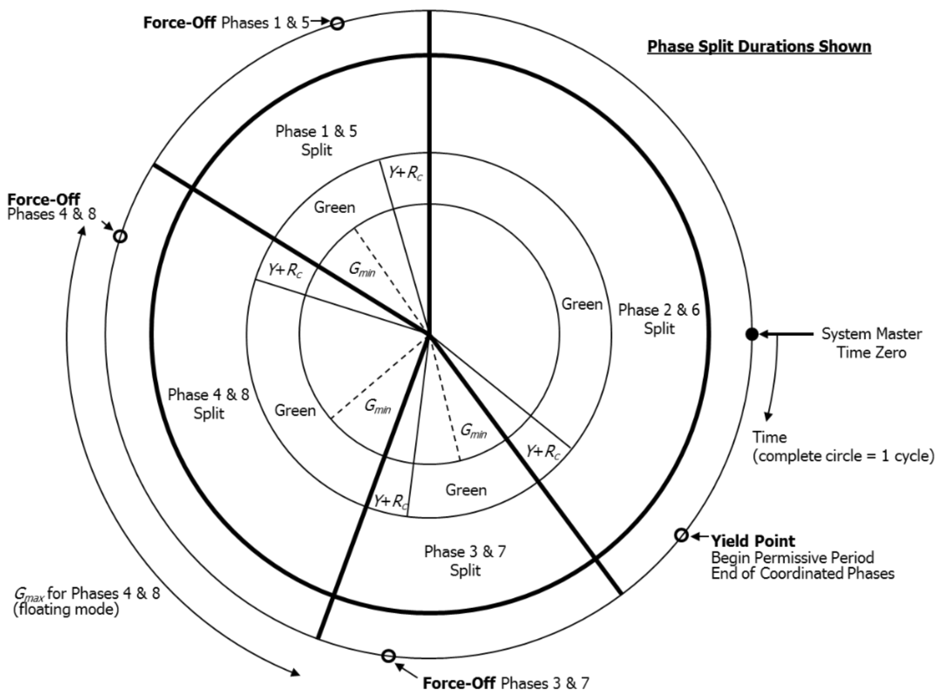
The relationship between the force-off points, yield point, and phase splits is shown in Exhibit 31-3. The yield point is associated with the coordinated phases (i.e., Phases 2 and 6). It coincides with the start of the yellow change interval. If a call for service by one of the noncoordinated phases arrives after the yield point is reached, then the coordinated phases begin the termination process by presenting the yellow indication. Calls that arrive before the yield point are not served until the yield point is reached.

The force-off and yield points for common phase pairs are shown in Exhibit 31-3 to occur at the same time. This approach is shown for convenience of illustration. In practice, the two phases may have different force-off or yield points.

A permissive period typically follows the yield point. If a conflicting call arrives during the permissive period, then the phase termination process begins immediately, and all phases associated with conflicting calls are served in sequence. Permissive periods are typically long enough to ensure that all calls for service are met during the signal cycle. This methodology does not explicitly model permissive periods. It is assumed the permissive period begins at the yield point and is sufficiently long that all conflicting calls are served in sequence each cycle.

One force-off point is associated with each of Phases 1, 3, 4, 5, 7, and 8. If a phase is extended to its force-off point, the phase begins the termination process by presenting the yellow indication (phases that terminate at a barrier must be in agreement to terminate before the yellow indication will be presented). Modern controllers compute the force-off points and yield point by using the entered phase splits and change periods. These computations are based on the relationships shown in Exhibit 31-3.

Exhibit 31-3
Force-Off Points, Yield Point, and Phase Splits



The concept of equivalent maximum green is useful for modeling noncoordinated phase operation. This maximum green replicates the effect of a force-off or yield point on phase duration. The procedure described in this subsection is used to compute the equivalent maximum green for coordinated-actuated operation. Separate procedures are described for the fixed force mode and the floating force mode.

A. Determine Equivalent Maximum Green for Floating Force Mode

This step is applicable if the controller is set to operate in the floating force mode. With this mode, each noncoordinated phase has its force-off point set at the split time after the phase first becomes active. The force-off point for a phase is established when the phase is first activated. Thus, the force-off point "floats," or changes, each time the phase is activated. This operation allows unused split time to revert to the coordinated phase via an early return to green. The equivalent maximum green for this mode is computed as being equal to the phase split less the change period. This relationship is shown in Exhibit 31-3 for Phases 4 and 8.

B. Determine Equivalent Maximum Green for Fixed Force Mode

This step is applicable if the controller is set to operate in the fixed force mode. With this mode, each noncoordinated phase has its force-off point set at a fixed time in the cycle relative to time zero on the system master. The force-off points are established whenever a new timing plan is selected (e.g., by time of day) and remains "fixed" until a new plan is selected. This operation allows unused split time to revert to the following phase.

The equivalent maximum green for this mode is computed for each phase by first establishing the fixed force-off points (as shown in Exhibit 31-3) and then computing the average duration of each noncoordinated phase. The calculation process is iterative. For the first iteration, the equivalent maximum green is set equal to the phase split less the change period. Thereafter, the equivalent maximum green for a specific phase is computed as the difference between its force-off point and the sum of the previous phase durations, starting with the first noncoordinated phase. Equation 31-27 illustrates this computation for Phase 4, using the ring structure shown in Exhibit 19-2. A similar calculation is performed for the other phases.

$$G_{max,4} = FO_4 - (YP_2 + CP_2 + G_3 + CP_3)$$

Equation 31-27

where

$G_{max,4}$ = equivalent maximum green for Phase 4 (s),

FO_4 = force-off point for Phase 4 (s),

YP_2 = yield point for Phase 2 (s),

G_3 = green interval duration for Phase 3 (s), and

CP_3 = change period (yellow change interval plus red clearance interval) for Phase 3 (s).

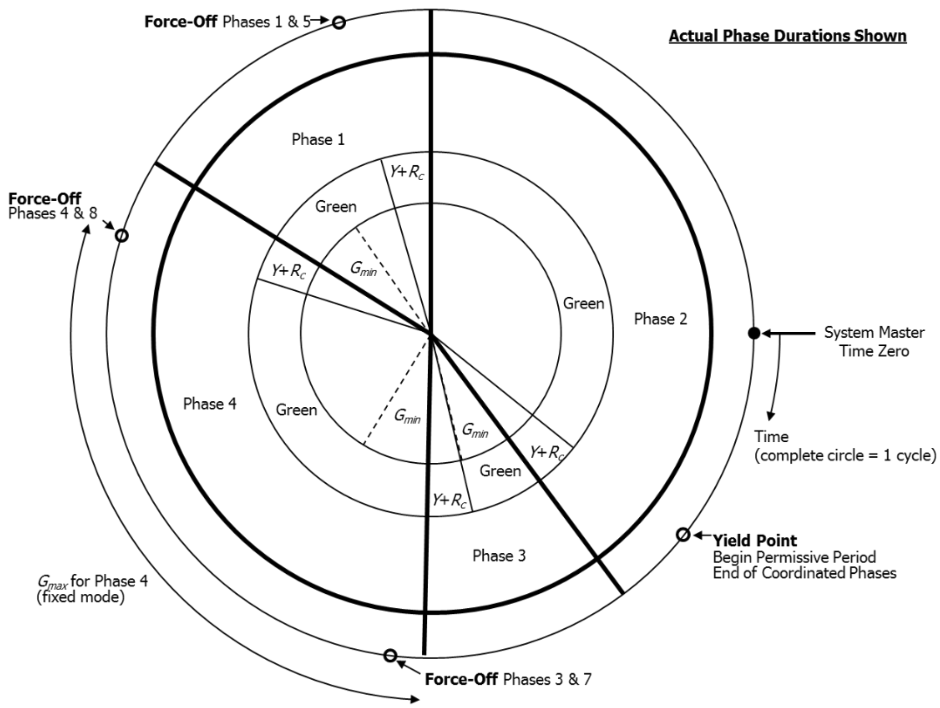
The maximum green obtained from Equation 31-27 is shown in Exhibit 31-4 for the ring that serves Phases 1, 2, 3, and 4. Unlike Exhibit 31-3, Exhibit 31-4 illustrates the *actual* average phase durations for a given cycle. In this example, Phase 3 timed to its minimum green and terminated. It never reached its force-off point. The unused time from Phase 3 was made available to Phase 4, which resulted in a larger maximum green than was obtained with the floating mode (see Exhibit 31-3). If every noncoordinated phase extends to its force-off point, then the maximum green from the fixed force mode equals that obtained from the floating force mode.

Average Phase Duration

This subsection describes the sequence of calculations needed to estimate the average duration of a phase. In fact, the process requires the combined calculation of the duration of all phases together because of the constraints imposed by the controller ring structure and associated barriers.

The calculation process is iterative because several intermediate equations require knowledge of the green interval duration. Specifically, the green interval duration is required in calculating lane group flow rate, queue service time, permitted green time, left-turn volume served during the permitted portion of a protected-permitted mode, and equivalent maximum green. To overcome this circular dependency, the green interval for each phase is initially estimated, and then the procedure is implemented by using this estimate. When completed, the procedure provides a new initial estimate of the green interval duration. The calculations are repeated until the initial estimate and computed green interval duration are effectively equal.

Exhibit 31-4
Example Equivalent Maximum Green for Fixed Force Mode



The calculation steps that constitute the procedure are described in the following paragraphs.

A. Compute Effective Change Period

The change period is computed for each phase. It is equal to the sum of the yellow change interval and the red clearance interval (i.e., $Y + R_c$). For phases that end at a barrier, the longer change period of the two phases that terminate at a barrier is used to define the effective change period for both phases.

B. Estimate Green Interval

An initial estimate of the green interval duration is provided for each phase. For the first iteration with fully actuated control, the initial estimate is equal to the maximum green setting. For the first iteration with coordinated-actuated control, the initial estimate is equal to the input phase split less the change period.

C. Compute Equivalent Maximum Green (Coordinated-Actuated)

If the controller is operating as coordinated-actuated, then the equivalent maximum green is computed for each phase. It is based on the estimated green interval duration, phase splits, and change periods. The previous subsection titled Equivalent Maximum Green describes how to compute this value.

D. Construct Queue Accumulation Polygon

The QAP is constructed for each lane group and corresponding phase by using the known flow rates and signal timing. The procedure for constructing this polygon is summarized in the previous subsection titled Queue Accumulation Polygon. It is described in more detail in Section 3.

E. Compute Queue Service Time

The queue service time g_s is computed for each QAP constructed in the previous step. For through movements or left-turn movements served during a left-turn phase, the polygon in Exhibit 31-1 applies and Equation 31-9 can be used. The procedure described in Section 3 is applicable to more complicated polygon shapes.

F. Compute Call Rate to Extend Green

The extending call rate is represented as the flow rate parameter λ. This parameter is computed for each lane group served by an actuated phase and is then aggregated to a phase-specific value. The procedure for computing this parameter is described in the previous subsection titled Volume Computations.

G. Compute Equivalent Maximum Allowable Headway

The equivalent maximum allowable headway MAH^* is computed for each actuated phase. The procedure for computing MAH^* is described in the previous subsection titled Maximum Allowable Headway.

H. Compute Number of Extensions Before Max-Out

The average number of extensions before the phase terminates by max-out is computed for each actuated phase with Equation 31-28.

$$n = q^*[G_{max} - (g_s + l_1)] \geq 0.0$$

Equation 31-28

where n is the number of extensions before the green interval reaches its maximum limit, G_{max} is the maximum green setting (s), and all other variables are as previously defined.

I. Compute Probability of Green Extension

The probability of the green interval being extended by randomly arriving vehicles is computed for each actuated phase with Equation 31-29.

$$p = 1 - \varphi^* e^{-\lambda^*(MAH^* - \Delta^*)}$$

Equation 31-29

where p is the probability of a call headway being less than the maximum allowable headway.

J. Compute Green Extension Time

The average green extension time is computed for each actuated phase with Equation 31-30.

Equation 31-30

$$g_e = \frac{p^2(1 - p^n)}{q^*(1 - p)}$$

K. Compute Activating Call Rate

The call rate to activate a phase is computed for each actuated phase. A separate rate is computed for vehicular traffic and for pedestrian traffic. The rate for each travel mode is based on its flow rate and the use of dual entry. The procedure for computing this rate is described in the previous subsection titled Volume Computations.

L. Compute Probability of Phase Call

The probability that an actuated phase is called depends on whether it is set on recall in the controller. If it is on recall, then the probability that the phase is called equals 1.0. If the phase is not on recall, then the probability that it is called can be estimated by using Equation 31-31 with Equation 31-32 and Equation 31-33.

Equation 31-31

$$p_c = p_v\left(1 - p_p\right) + p_p(1 - p_v) + p_v\, p_p$$

with

Equation 31-32

$$p_v = 1 - e^{-q_v^*\, C}$$

Equation 31-33

$$p_p = 1 - e^{-q_p^*\, P_p\, C}$$

where

p_c = probability that the subject phase is called,

p_v = probability that the subject phase is called by a vehicle detection,

p_p = probability that the subject phase is called by a pedestrian detection,

q_v^* = activating vehicular call rate for the phase (veh/s),

q_p^* = activating pedestrian call rate for the phase (p/s), and

P_p = probability of a pedestrian pressing the detector button = 0.51.

The probability of a pedestrian pressing the detector button reflects the tendency of some pedestrians to decline from using the detector button before crossing a street. Research indicates about 51% of all crossing pedestrians will push the button to place a call for pedestrian service (*1*).

M. Compute Unbalanced Green Duration

The unbalanced average green interval duration is computed for each actuated phase by using Equation 31-34 with Equation 31-35 and Equation 31-36.

Equation 31-34

$$G_u = G_{|veh,call}\, p_v\left(1 - p_p\right) + G_{|ped,call}\, p_p(1 - p_v) + \max\left(G_{|veh,call}, G_{|ped,call}\right) p_v\, p_p \le G_{max}$$

with

Equation 31-35

$$G_{|veh,call} = \max\ \left(l_1 + g_s + g_e, G_{min}\right)$$

$$G_{|ped,call} = Walk + PC$$

Equation 31-36

where

G_u = unbalanced green interval duration for a phase (s),

$G_{|veh,\text{call}}$ = average green interval given that the phase is called by a vehicle detection (s),

G_{min} = minimum green setting (s),

$G_{|ped,\text{call}}$ = average green interval given that the phase is called by a pedestrian detection (s),

Walk = pedestrian walk setting (s), and

PC = pedestrian clear setting (s).

If maximum recall is set for the phase, then G_u is equal to G_{max}. If the phase serves a left-turn movement that operates in the protected mode, then the probability that it is called by pedestrian detection p_p is equal to 0.0.

If the phase serves a left-turn movement that operates in the protected-permitted mode and the left-turn movement shares a lane with through vehicles, then the green interval duration is equal to the phase's minimum green setting.

The green interval duration obtained from this step is "unbalanced" because it does not reflect the constraints imposed by the controller ring structure and associated barriers. These constraints are imposed in Step O or Step P, depending on the type of control used at the intersection.

It is assumed the rest-in-walk mode is not enabled.

N. Compute Unbalanced Phase Duration

The unbalanced average phase duration is computed for each actuated phase by adding the unbalanced green interval duration and the corresponding change period components. This calculation is completed with Equation 31-37.

$$D_{up} = G_u + Y + R_c$$

Equation 31-37

where D_{up} is the unbalanced phase duration (s).

If simultaneous gap-out is enabled, the phase ends at a barrier, and the subject phase experiences green extension when the concurrent phase has reached its maximum green limit, then both phases are extended, but only due to the call flow rate of the subject phase. Hence, the green extension time computed in Step J is too long. The effect is accounted for in the current step by multiplying the green extension time from Step J by a "flow rate ratio." This ratio represents the sum of the flow rate parameter for each lane group served by the subject phase divided by the sum of the flow rate parameter for each group served by the subject phase and served by the concurrent phase (the latter sum equals the call rate from Step F).

O. Compute Average Phase Duration—Fully Actuated Control

For this discussion, it is assumed Phases 2 and 6 are serving Movements 2 and 6, respectively, on the major street (see Exhibit 19-2). If the left-turn movements on the major street operate in the protected mode or the protected-permitted mode, then Movements 1 and 5 are served during Phases 1 and 5, respectively. Similarly, Phases 4 and 8 are serving Movements 4 and 8, respectively, on the minor street. If the left-turn movements on the minor street are protected or protected-permitted, then Phases 3 and 7 are serving Movements 3 and 7, respectively. If a through movement phase occurs first in a phase pair, then the other phase (i.e., the one serving the opposing left-turn movement) is a lagging left-turn phase.

The following rules are used to estimate the average duration of each phase:

1. Given two phases that occur in sequence between barriers (i.e., phase *a* followed by phase *b*), the duration of $D_{p,a}$ is equal to the unbalanced phase duration of the first phase to occur (i.e., $D_{p,a} = D_{up,a}$). The duration of $D_{p,b}$ is based on Equation 31-38 for the major-street phases.

Equation 31-38

$$D_{p,b} = \max\left(D_{up,1} + D_{up,2}, D_{up,5} + D_{up,6}\right) - D_{p,a}$$

where

$D_{p,b}$ = phase duration for phase *b*, which occurs just after phase *a* (s);

$D_{p,a}$ = phase duration for phase *a*, which occurs just before phase *b* (s); and

$D_{up,i}$ = unbalanced phase duration for phase *i*; i = 1, 2, 5, and 6 for major street, and i = 3, 4, 7, and 8 for minor street (s).

Equation 31-39 applies for the minor-street phases.

Equation 31-39

$$D_{p,b} = \max\left(D_{up,3} + D_{up,4}, D_{up,7} + D_{up,8}\right) - D_{p,a}$$

For example, if the phase pair consists of Phase 3 followed by Phase 4 (i.e., a leading left-turn arrangement), then $D_{p,3}$ is set to equal $D_{up,3}$ and $D_{p,4}$ is computed from Equation 31-39. In contrast, if the pair consists of Phase 8 followed by Phase 7 (i.e., a lagging left-turn arrangement), then $D_{p,8}$ is set to equal $D_{up,8}$ and $D_{p,7}$ is computed from Equation 31-39.

2. If an approach is served with one phase operating in the permitted mode (but not split phasing), then $D_{p,a}$ equals 0.0, and the equations above are used to estimate the duration of the phase (i.e., $D_{p,b}$).

3. If split phasing is used, then $D_{p,a}$ equals the unbalanced phase duration for one approach and $D_{p,b}$ equals the unbalanced phase duration for the other approach.

P. Compute Average Phase Duration—Coordinated-Actuated Control

For this discussion, it is assumed Phases 2 and 6 are the coordinated phases serving Movements 2 and 6, respectively (see Exhibit 19-2). If the left-turn movements operate in the protected mode or the protected-permitted mode, then the opposing left-turn movements are served during Phases 1 and 5. If a coordinated phase occurs first in the phase pair, then the other phase (i.e., the one serving the opposing left-turn movement) is a lagging left-turn phase.

The following rules are used to estimate the average duration of each phase:

1. If the phase is associated with the street serving the coordinated movements, then
   a. If a left-turn phase exists for the subject approach, then its duration $D_{p,l}$ equals $D_{up,l}$, and the opposing through phase has a duration $D_{p,t}$, which is calculated by using Equation 31-40.

      $$D_{p,t} = C - \max(D_{up,3} + D_{up,4}, D_{up,7} + D_{up,8}) - D_{p,l}$$

      Equation 31-40

      where $D_{p,t}$ is the phase duration for coordinated phase t (t = 2 or 6) (s), $D_{p,l}$ is the phase duration for left-turn phase l (l = 1 or 5) (s), and all other variables are as previously defined.

      If Equation 31-40 is applied to Phase 2, then t equals 2 and l equals 1. If it is applied to Phase 6, then t equals 6 and l equals 5.

   b. If a left-turn phase does not exist for the subject approach, then $D_{p,l}$ equals 0.0, and Equation 31-40 is used to estimate the duration of the coordinated phase.

   This procedure for determining average phase duration accommodates split phasing only on the street that does not serve the coordinated movements.

   If $D_{p,t}$ obtained from Equation 31-40 is less than the minimum phase duration (= $G_{min} + Y + R_c$), then the phase splits are too generous and do not leave adequate time for the coordinated phases.

2. If the phase is associated with the street serving the noncoordinated movements, then the rules described in Step O are used to determine the phase's average duration.

Q. Compute Green Interval Duration

The average green interval duration is computed for each phase by subtracting the yellow change and red clearance intervals from the average phase duration.

$$G = D_p - Y - R_c$$

Equation 31-41

where G is the green interval duration (s).

R. Compare Computed and Estimated Green Interval Durations

The green interval duration from the previous step is compared with the value estimated in Step B. If the two values differ by 0.1 s or more, then the computed green interval becomes the new initial estimate, and the sequence of calculations is repeated starting with Step C. This process is repeated until the two green intervals differ by less than 0.1 s.

If the intersection is semiactuated or fully actuated, then the equilibrium cycle length is computed with Equation 31-42.

$$C_e = \sum_{i=1}^{4} D_{p,i}$$

Equation 31-42

where C_e is the equilibrium cycle length (s) and i is the phase number. The sum in this equation includes all phases in Ring 1. The equilibrium cycle length is used in all subsequent calculations in which cycle length C is an input variable.

Probability of Max-Out

When the green indication is extended to its maximum green limit, the associated phase is considered to have terminated by max-out. The probability of max-out provides useful information about phase performance. When max-out occurs, the phase ends without consideration of whether the queue is served or vehicles are in the dilemma zone. Hence, a phase that frequently terminates by max-out may have inadequate capacity and may be associated with more frequent rear-end crashes.

The probability of max-out can be equated to the joint probability of there being a sequence of calls to the phase in service, each call having a headway that is shorter than the equivalent maximum allowable headway for the phase. This probability can be stated mathematically by using Equation 31-43 with Equation 31-44 and Equation 31-45.

Equation 31-43

$$p_x = p^{n_x}$$

with

Equation 31-44

$$n_x = \frac{G_{max} - MAH^* - (g_s + l_1)}{h} \geq 0.0$$

Equation 31-45

$$h = \frac{\Delta^* + (\varphi^*/\lambda^*) - (MAH^* + [1/\lambda^*]\varphi^* e^{-\lambda^*(MAH^* - \Delta^*)}}{1 - \varphi^* e^{-\lambda^*(MAH^* - \Delta^*)}}$$

where

p_x = probability of phase termination by extension to the maximum green limit,

h = average call headway for all calls with headways less than MAH^* (s), and

n_x = number of calls necessary to extend the green to max-out.

LANE GROUP FLOW RATE ON MULTIPLE-LANE APPROACHES

Introduction

When drivers approach an intersection, their primary criterion for lane choice is movement accommodation (i.e., left, through, or right). If multiple exclusive lanes are available to accommodate their movement, they tend to choose the lane that minimizes their service time (i.e., the time required to reach the stop line, as influenced by the number and type of vehicles between them and the stop line). This criterion tends to result in relatively equal lane use under most circumstances.

If one of the lanes being considered is a shared lane, then service time is influenced by the distribution of turning vehicles in the shared lane. Turning vehicles tend to have a longer service time because of the turn maneuver. Moreover, when turning vehicles operate in the permitted mode, their service time can be lengthy because of the gap search process.

Observation of driver lane-choice behavior indicates there is an equilibrium lane flow rate that characterizes the collective choices of the population of drivers. Research indicates the equilibrium flow rate can be estimated from the lane volume distribution that yields the minimum service time for the population of drivers having a choice of lanes (*2*).

A model for predicting the equilibrium lane flow rate on an intersection approach is described in this subsection. The model is based on the principle that through drivers will choose the lane that minimizes their perceived service time. As a result of this lane selection process, each lane will have the same minimum service time. The principle is represented mathematically by (*a*) defining service time for each lane as the product of lane flow rate and saturation headway, (*b*) representing this product as the lane demand–to–saturation flow rate ratio (i.e., *v*/*s* ratio), and (*c*) making the *v*/*s* ratios equal among alternative approach lanes. Equation 31-46 is derived from this representation.

$$\frac{v_i}{s_i} = \frac{\sum_{i=1}^{N_{th}} v_i}{\sum_{i=1}^{N_{th}} s_i}$$

Equation 31-46

where

v_i = demand flow rate in lane i (veh/h/ln),

s_i = saturation flow rate in lane i (veh/h/ln), and

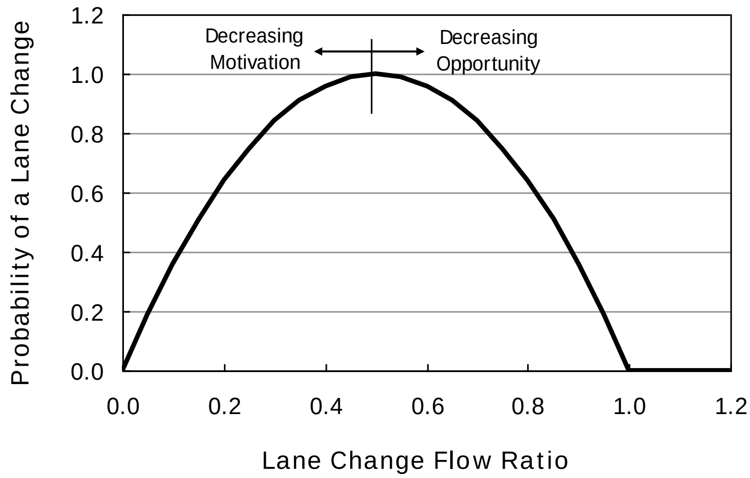
N_{th} = number of through lanes (shared or exclusive) (ln).

The "equalization of flow ratios" principle has been embodied in the HCM since the 1985 edition. Specifically, it has been used to derive the equation for estimating the proportion of left-turning vehicles in a shared lane P_L.

During field observations of various intersection approaches, it was noted that the principle overestimated the effect of turning vehicles in shared lanes for very low and for very high approach flow-rate conditions (*3*). Under low flow-rate conditions, it was rationalized that through drivers are not motivated to change lanes because the frequency of turns is very low and the threat of delay is negligible. Under high flow-rate conditions, it was rationalized that through drivers do not have an opportunity to change lanes because of the lack of adequate gaps in the outside lane. The field observations also indicated that most lane choice decisions (and related lane changes) for through drivers tended to occur upstream of the intersection, before deceleration occurs.

As a result of these field observations (*3*), the model was extended to include the probability of a lane change. The probability of a lane change represents the joint probability of there being motivation (i.e., moderate to high flow rates) and opportunity (i.e., adequate lane-change gaps). A variable that is common to each probability distribution is the ratio of the approach flow rate to the maximum flow rate that would allow any lane changes. This maximum flow rate is the rate corresponding to the minimum headway considered acceptable for a lane change (i.e., about 3.7 s) (*4*). Exhibit 31-5 illustrates the modeled relationship between lane change probability and the flow ratio in the traffic lanes upstream of the intersection, before deceleration occurs (*3*).

Exhibit 31-5
Probability of a Lane Change



Procedure

The procedure described in this subsection is generalized so it can be applied to any signalized intersection approach with any combination of exclusive turn lanes, shared lanes, and exclusive through lanes. At least one shared lane must be present, and the approach must have two or more lanes (or bays) serving two or more traffic movements. This type of generalized formulation is attractive because of its flexibility; however, the trade-off is that the calculation process is iterative. If a closed-form solution is desired, then one would likely have to be uniquely derived for each lane assignment combination.

The procedure is described in the following steps. Input variables used in the procedure are identified in the following list and are shown in Exhibit 31-6:

N_l = number of lanes in exclusive left-turn lane group (ln),

N_{sl} = number of lanes in shared left-turn and through lane group (ln),

N_t = number of lanes in exclusive through lane group (ln),

N_{sr} = number of lanes in shared right-turn and through lane group (ln),

N_r = number of lanes in exclusive right-turn lane group (ln),

N_{lr} = number of lanes in shared left- and right-turn lane group (ln),

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

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

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

v_l = demand flow rate in exclusive left-turn lane group (veh/h/ln),

v_{sl} = demand flow rate in shared left-turn and through lane group (veh/h),

v_t = demand flow rate in exclusive through lane group (veh/h/ln),

v_{sr} = demand flow rate in shared right-turn and through lane group (veh/h),

v_r = demand flow rate in exclusive right-turn lane group (veh/h/ln),

v_{lr} = demand flow rate in shared left- and right-turn lane group (veh/h),

$v_{sl,lt}$ = left-turn flow rate in shared lane group (veh/h/ln),

$v_{sr,rt}$ = right-turn flow rate in shared lane group (veh/h/ln),

s_l = saturation flow rate in exclusive left-turn lane group with permitted operation (veh/h/ln),

s_{sl} = saturation flow rate in shared left-turn and through lane group with permitted operation (veh/h/ln),

s_t = saturation flow rate in exclusive through lane group (veh/h/ln),

s_{sr} = saturation flow rate in shared right-turn and through lane group with permitted operation (veh/h/ln),

s_r = saturation flow rate in exclusive right-turn lane group with permitted operation (veh/h/ln),

s_{lr} = saturation flow rate in shared left- and right-turn lane group (veh/h/ln),

s_{th} = saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, area type, work zone presence, downstream lane blockage, and spillback) (veh/h/ln),

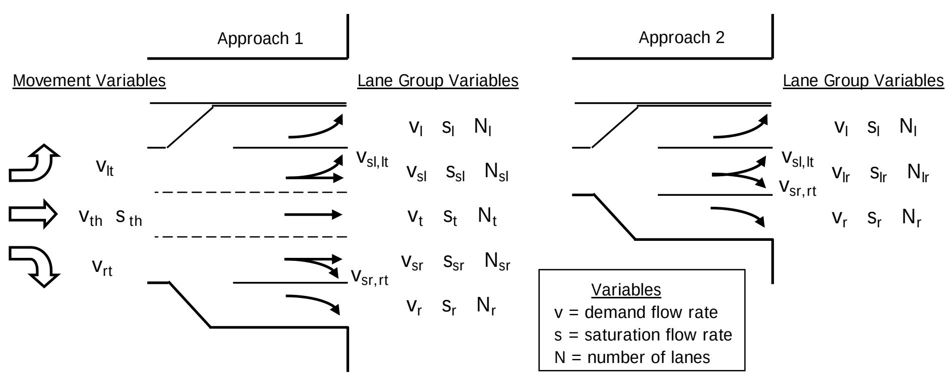
g_p = effective green time for permitted left-turn operation (s),

g_f = time before the first left-turning vehicle arrives and blocks the shared lane (s), and

g_u = duration of permitted left-turn green time that is not blocked by an opposing queue (s).

Each shared-lane lane group has one lane (i.e., $N_{sl} = 1$, $N_{sr} = 1$, and $N_{lr} = 1$). Procedures for calculating g_p, g_f, and g_u are provided in Section 3.

Exhibit 31-6
Input Variables for Lane Group Flow Rate Procedure



A. Compute Modified Through-Car Equivalents

Three modified through-car equivalent factors are computed for the left-turn movement. These factors are computed with Equation 31-47 through Equation 31-51.

$$E_{L,m} = (E_L - 1)P_{lc} + 1$$

Equation 31-47

Equation 31-48

$$E_{L1,m} = \left(\frac{E_{L1}}{f_{Lpb}} - 1\right) P_{lc} + 1$$

Equation 31-49

$$E_{L2,m} = \left(\frac{E_{L2}}{f_{Lpb}} - 1\right) P_{lc} + 1$$

with

Equation 31-50

$$P_{lc} = 1 - \left(\left[2\frac{v_{app}}{s_{lc}}\right] - 1\right)^2 \geq 0.0$$

Equation 31-51

$$v_{app} = \frac{v_{lt} + v_{th} + v_{rt}}{N_{sl} + N_t + N_{sr}}$$

where

$E_{L,m}$ = modified through-car equivalent for a protected left-turning vehicle,

$E_{L1,m}$ = modified through-car equivalent for a permitted left-turning vehicle,

E_L = equivalent number of through cars for a protected left-turning vehicle (= 1.05),

E_{L1} = equivalent number of through cars for a permitted left-turning vehicle,

$E_{L2,m}$ = modified through-car equivalent for a permitted left-turning vehicle when opposed by a queue on a single-lane approach,

E_{L2} = equivalent number of through cars for a permitted left-turning vehicle when opposed by a queue on a single-lane approach,

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

P_{lc} = probability of a lane change among the approach through lanes,

v_{app} = average demand flow rate per through lane (upstream of any turn bays on the approach) (veh/h/ln),

s_{lc} = maximum flow rate at which a lane change can occur = 3,600/t_{lc} (veh/h/ln), and

t_{lc} = critical merge headway = 3.7 (s).

The factor obtained from Equation 31-49 is applicable when permitted left-turning vehicles are opposed by a queue on a single-lane approach. Equations for calculating E_{L1} and E_{L2} are provided in Section 3. A procedure for calculating f_{Lpb} is provided later in this section.

If the approach has a shared left- and right-turn lane (as shown in Approach 2 in Exhibit 31-6), then Equation 31-52 is used to compute the average demand flow rate per lane (with N_{lr} = 1.0).

Equation 31-52

$$v_{app} = (v_{lt} + v_{rt})/N_{lr}$$

The modified through-car equivalent for permitted right-turning vehicles is computed with Equation 31-53.

Equation 31-53

$$E_{R,m} = \left(\frac{E_R}{f_{Rpb}} - 1\right) P_{lc} + 1$$

where $E_{R,m}$ is the modified through-car equivalent for a protected right-turning vehicle, f_{Rpb} is the pedestrian–bicycle adjustment factor for right-turn groups, E_R is the equivalent number of through cars for a protected right-turning vehicle (= 1.18), and all other variables are as previously defined.

A procedure for calculating f_{Rpb} is provided later in this section.

If the opposing approach has two lanes serving through vehicles and the inside lane serves through and left-turn vehicles, then Equation 31-54 is used to compute the adjusted duration of permitted left-turn green time that is not blocked by an opposing queue g_u^*. This variable is then used in Equation 31-59 in replacement of the variable g_u. This adjustment is intended to reflect the occasional hesitancy of drivers to shift from the inside lane to the outside lane during higher-volume conditions for this approach-lane geometry. In all other cases of opposing approach-lane geometry, the variable g_u^* is not computed and Equation 31-59 is used as described in the text.

$$g_u^* = g_u + (g_{diff} \times P_{lc})$$

Equation 31-54

where

g_u^* = adjusted duration of permitted left-turn green time that is not blocked by an opposing queue (s), and

g_{diff} = supplemental service time (s).

Equation 31-107 in Section 3 can be used to calculate g_{diff}.

B. Estimate Shared-Lane Lane Group Flow Rate

The procedure to estimate the shared-lane lane group flow rate requires an initial estimate of the demand flow rate for each traffic movement in each shared-lane lane group on the subject approach. For the shared lane serving left-turn and through vehicles, the left-turn flow rate in the shared lane $v_{sl,lt}$ is initially estimated as 0.0 veh/h, and the total lane group flow rate v_{sl} is estimated as equal to the average flow rate per through lane v_{app}. For the shared lane serving right-turn vehicles, the right-turn flow rate in the shared lane $v_{sr,rt}$ is estimated as 0.0 veh/h, and the total lane group flow rate v_{sr} is estimated as equal to the average flow rate per through lane v_{app}. These estimates are updated in a subsequent step.

C. Compute Exclusive Lane-Group Flow Rate

The demand flow rate in the exclusive left-turn lane group v_l is computed with Equation 31-55, where all variables are as previously defined.

$$v_l = \frac{v_{lt} - v_{sl,lt}}{N_l} \geq 0.0$$

Equation 31-55

A similar calculation is completed to estimate the demand flow rate in the exclusive right-turn lane group v_r. The flow rate in the exclusive through lane group is then computed with Equation 31-56.

$$v_t = \frac{v_{th} - (v_{sl} - v_{sl,lt}) - (v_{sr} - v_{sr,rt})}{N_t} \geq 0.0$$

Equation 31-56

D. Compute Proportion of Turns in Shared-Lane Lane Groups

The proportion of left-turning vehicles in the shared left-turn and through lane is computed with Equation 31-57.

Equation 31-57

$$P_L = \frac{v_{sl,lt}}{v_{sl}} \leq 1.0$$

where P_L is the proportion of left-turning vehicles in the shared lane. Substitution of $v_{sr,rt}$ for $v_{sl,lt}$ and v_{sr} for v_{sl} in Equation 31-57 yields an estimate of the proportion of right-turning vehicles in the shared lane P_R.

The proportion of left-turning vehicles in the shared left- and right-turn lane is computed with Equation 31-58.

Equation 31-58

$$P_L = \frac{v_{sl,lt}}{v_{lr}} \leq 1.0$$

Substituting $v_{sr,rt}$ for $v_{sl,lt}$ in Equation 31-58 yields an estimate of the proportion of right-turning vehicles in the shared lane P_R.

E. Compute Lane Group Saturation Flow Rate

The saturation flow rate for the lane group shared by the left-turn and through movements is computed by using Equation 31-59 with Equation 31-60.

Equation 31-59

$$s_{sl} = \frac{s_{th}}{g_p}\left(g_f + \frac{g_{diff}}{1 + P_L\left[E_{L2,m} - 1\right]} + \frac{\min\left[g_p - g_f, g_u\right]}{1 + P_L\left[E_{L1,m} - 1\right]} + \frac{3{,}600\, n_s^*\, f_{ms}\, f_{sp}}{s_{th}}\right)$$

with

Equation 31-60

$$n_s^* = \begin{cases} \dfrac{P_L}{1 - P_L}\left(1 - P_L^{n_s}\right) & \text{if } P_L < 0.999 \\ n_s\, P_L & \text{if } P_L \geq 0.999 \end{cases}$$

where g_{diff} is the supplemental service time (s), n_s^* is the expected number of sneakers per cycle in a shared left-turn lane, f_{ms} is the adjustment factor for downstream lane blockage, f_{sp} is the adjustment factor for sustained spillback, and all other variables are as previously defined.

Equation 31-107 in Section 3 can be used to calculate g_{diff}.

Equation 31-61 is used to compute the saturation flow rate in a shared right-turn and through lane group s_{sr}.

Equation 31-61

$$s_{sr} = \frac{s_{th}}{1 + P_R(E_{R,m} - 1)}$$

where P_R is the proportion of right-turning vehicles in the shared lane (decimal).

The saturation flow rate for the lane group serving left-turning vehicles in an exclusive lane s_l is computed with Equation 31-59, with P_L = 1.0, g_{diff} = 0.0, g_f = 0.0, and s_{th} replaced by s_{lt} (see Equation 31-112). Similarly, the saturation flow rate in an exclusive right-turn lane group s_r is computed with Equation 31-61, with P_R = 1.0.

The saturation flow rate for the lane group serving through vehicles in an exclusive lane is computed with Equation 31-62.

$$s_t = s_{th}\, f_s$$ Equation 31-62

where f_s is the adjustment factor for all lanes serving through vehicles on an approach with a shared left-turn and through lane group (= 1.0 if N_{sl} = 0; 0.91 otherwise).

The saturation flow rate for the shared left- and right-turn lane is computed with Equation 31-63.

$$s_{lr} = \frac{s_{th}}{1 + P_L(E_{L,m} - 1) + P_R(E_{R,m} - 1)}$$ Equation 31-63

F. Compute Flow Ratio

The flow ratio for the subject intersection approach is computed with Equation 31-64.

$$y^* = \frac{v_l\, N_l + v_{sl}\, N_{sl} + v_t\, N_t + v_{sr}\, N_{sr} + v_r\, N_r + v_{lr}\, N_{lr}}{s_l\, N_l + s_{sl}\, N_{sl} + s_t\, N_t + s_{sr}\, N_{sr} + s_r\, N_r + s_{lr}\, N_{lr}}$$ Equation 31-64

where y^* is the flow ratio for the approach. If a shared left- and right-turn lane exists on the subject approach, then $N_{sl} = 0$, $N_t = 0$, $N_{sr} = 0$, and $N_{lr} = 1$; otherwise, $N_{sl} = 1$, $N_t \geq 0$, $N_{sr} = 1$, and $N_{lr} = 0$.

G. Compute Revised Lane Group Flow Rate

The flow ratio from Step F is used to compute the demand flow rate in the exclusive left-turn lane group with Equation 31-65.

$$v_l = s_l\, y^*$$ Equation 31-65

In a similar manner, the demand flow rate for the other lane groups is estimated by multiplying the flow ratio y^* by the corresponding lane group saturation flow rate.

H. Compute Turn Movement Flow Rate in Shared-Lane Lane Groups

The left-turn demand flow rate in the shared lane group is computed with Equation 31-66.

$$v_{sl,lt} = v_{lt} - v_l \geq 0.0$$ Equation 31-66

Equation 31-66 can be used to compute the right-turn demand flow rate in the shared lane group by substituting $v_{sr,rt}$ for $v_{sl,lt}$, v_{rt} for v_{lt}, and v_r for v_l.

The demand flow rate in each shared-lane lane group is now compared with the rate estimated in Step B. If they differ by less than 0.1 veh/h, then the procedure is complete and the flow rates estimated in Steps G and H represent the best estimate of the flow rate for each lane group.

If there is disagreement between the lane group demand flow rates, then the calculations are repeated, starting with Step C. However, for this iteration, the flow rates computed in Steps G and H are used in the new calculation sequence. The calculations are complete when the flow rates used at the start of Step C differ from those obtained in Step H by less than 0.1 veh/h.

PRETIMED PHASE DURATION

The design of a pretimed timing plan can be a complex and iterative process that is generally carried out with the assistance of software. Several software products are available for this purpose. This subsection describes various strategies for pretimed signal-timing design and provides a procedure for implementing one of these strategies.

Design Strategies

Several aspects of signal-timing design, such as the choice of the timing strategy, are beyond the scope of this manual. Three basic strategies are commonly used for pretimed signals.

One strategy is to equalize the volume-to-capacity ratios for critical lane groups. It is the simplest strategy and the only one that can be calculated without excessive iteration. Under this strategy, the green time is allocated among the various signal phases in proportion to the flow ratio of the critical lane group for each phase. This strategy is described briefly in the next subsection. It is also used in the planning-level analysis application described in Section 5.

A second strategy is to minimize the total delay to all vehicles. This strategy is generally proposed as the optimal solution to the signal-timing problem. Variations of this strategy often combine other performance measures (e.g., stop rate, fuel consumption) in the optimization function. Many signal-timing software products offer this optimization feature. Some products use a delay estimation procedure identical to that in the motorized vehicle methodology in Chapter 19, but other products use minor departures from it.

A third strategy is to equalize the level of service (LOS) for all critical lane groups. This strategy promotes a LOS on all approaches that is consistent with the overall intersection LOS. It improves on the first and second strategies because they tend to produce a higher delay per vehicle for the minor movements at the intersection (and therefore a less favorable LOS).

Determining Phase Duration on the Basis of Vehicle Demand

Signal timing based on equalization of the volume-to-capacity ratio is described in this subsection. Equation 31-67, Equation 31-68, and Equation 31-69 are used to estimate the cycle length and effective green time for each critical phase. Conversion to green interval duration follows by applying the appropriate lost-time increments.

Equation 31-67

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

Equation 31-68

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

Equation 31-69

$$g_i = \frac{v_i\,C}{N_i\,s_i\,X_i} = \left(\frac{v}{N\,s}\right)_i \left(\frac{C}{X_i}\right)$$

where

C = cycle length (s),

L = cycle lost time (s),

X_c = critical intersection volume-to-capacity ratio,

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

ci = set of critical phases on the critical path,

X_i = volume-to-capacity ratio for lane group i,

v_i = demand flow rate for lane group i (veh/h),

N_i = number of lanes in lane group i (ln),

s_i = saturation flow rate for lane group i (veh/h/ln), and

g_i = effective green time for lane group i (s).

The summation term in each of these equations represents the summation of a specific variable for the set of critical phases. A critical phase is one phase of a set of phases that occurs in sequence whose combined flow ratio is the largest for the signal cycle.

Procedure

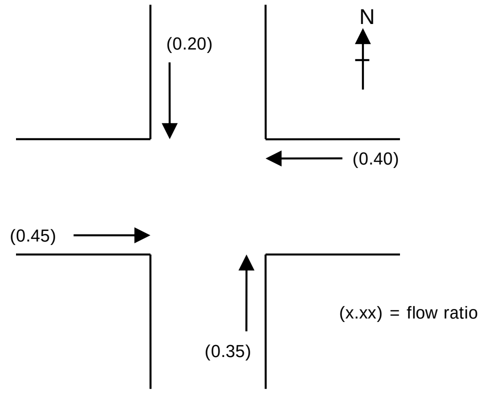
The following steps summarize the procedure for estimating the cycle length and effective green time for the critical phases:

1. Compute the flow ratio [= $v_i/(N\ s_i)$] for each lane group and identify the critical flow ratio for each phase. When there are several lane groups on the approach and they are served during a common phase, then the lane group with the largest flow ratio represents the critical flow ratio for the phase. A procedure for identifying the critical phases and associated flow ratios is described in Section 4 of Chapter 19, Signalized Intersections.
2. If signal-system constraints do not dictate the cycle length, then estimate the minimum cycle length with Equation 31-68 by setting X_c equal to 1.0.
3. If signal-system constraints do not dictate the cycle length, then estimate the desired cycle length with Equation 31-68 by substituting a target volume-to-capacity ratio X_t for the critical ratio X_c. A value of X_t in the range of 0.80 to 0.90 is recommended for this purpose.
4. If signal-system constraints do not dictate the cycle length, then use the results of Steps 2 and 3 to select an appropriate cycle length for the signal. Otherwise, the cycle length is that dictated by the signal system.
5. Estimate the effective green time for each phase with Equation 31-69 and the target volume-to-capacity ratio.
6. Check the timing to ensure the effective green time and the lost time for each phase in a common ring sum to the cycle length.

Example Application

The procedure is illustrated by a sample calculation. Consider the intersection shown in Exhibit 31-7.

Exhibit 31-7
Example Intersection



Phases 2 and 6 serve the eastbound and westbound approaches, respectively. Phases 4 and 8 serve the southbound and northbound approaches, respectively. One phase from each pair will represent the critical phase and dictate the duration of both phases. It is assumed the lost time for each phase equals the change period (i.e., the yellow change interval plus the red clearance interval). Thus, the lost time for each critical phase is 4 s, or 8 s for the cycle.

In this simple example, only one lane group is served on each approach, so the critical flow ratios can be identified by inspection of Exhibit 31-7. Specifically, the critical flow ratio for the east–west phases is that associated with the eastbound approach (i.e., Phase 2) at a value of 0.45. Similarly, the critical flow ratio for the north–south phases is that associated with the northbound approach (i.e., Phase 8).

The minimum cycle length that will avoid oversaturation is computed by Equation 31-68 with X_c = 1.00.

$$C(\text{minimum}) = \frac{8(1.0)}{1.0 - (0.45 + 0.35)} = \frac{8}{0.2} = 40 \text{ s}$$

A target volume-to-capacity ratio of 0.80 is used to estimate the target cycle length.

$$C = \frac{8(0.8)}{0.8 - (0.45 + 0.35)} = \frac{6.4}{0} = \text{infinity}$$

This computation indicates a critical volume-to-capacity ratio of 0.8 cannot be provided with the present demand levels at the intersection.

As a second trial estimate, a target volume-to-capacity ratio of 0.92 is selected and used to estimate the target cycle length.

$$C = \frac{8(0.92)}{0.92 - (0.45 + 0.35)} = 61 \text{ s}$$

The estimate is rounded to 60 s for practical application. Equation 31-67 is then used to estimate the critical volume-to-capacity ratio of 0.923 for the selected cycle length of 60 s.

With Equation 31-69, the effective green time is allocated so the volume-to-capacity ratio for each critical lane group is equal to the target volume-to-capacity ratio. Thus, for the example problem, the target volume-to-capacity ratio for each phase is 0.923. The effective green times are computed with Equation 31-69. The results of the calculations are listed below:

$$g_2 = 0.45(60/0.923) = 29.3 \text{ s}$$

$$g_8 = 0.35(60/0.923) = 22.7 \text{ s}$$

$$g_2 + g_8 + L = 29.3 + 22.7 + 8.0 = 60.0 \text{ s}$$

The duration of the effective green interval for Phase 6 is the same as for Phase 2, given that they have the same phase lost time. Similarly, the effective green interval for Phase 4 is the same as for Phase 8.

Determining Phase Duration on the Basis of Pedestrian Considerations

Two pedestrian considerations are addressed in this subsection as they relate to pretimed phase duration. One consideration addresses the time a pedestrian needs to perceive the signal indication and traverse the crosswalk. A second consideration addresses the time needed to serve cyclic pedestrian demand. When available, local guidelines or practice should be used to establish phase duration on the basis of pedestrian considerations.

A minimum green interval duration that allows a pedestrian to perceive the indication and traverse the crosswalk can be computed with Equation 31-70.

$$G_{p,min} = t_{pr} + \frac{L_{cc}}{S_p} - Y - R_c$$

Equation 31-70

where

- $G_{p,min}$ = minimum green interval duration based on pedestrian crossing time (s),
- t_{pr} = pedestrian perception of signal indication and curb departure time = 7.0 (s),
- L_{cc} = curb-to-curb crossing distance (ft),
- S_p = pedestrian walking speed = 3.5 (ft/s),
- Y = yellow change interval (s), and
- R_c = red clearance interval (s).

The variable t_{pr} in this equation represents the time pedestrians need to perceive the start of the phase and depart from the curb. A value of 7.0 s represents a conservatively long value that is adequate for most pedestrian crossing conditions. The variable S_p represents the pedestrian walking speed in a crosswalk. A value of 3.5 ft/s represents a conservatively slow value that most pedestrians will exceed.

If a permitted or protected-permitted left-turn operation is used for the left-turn movement that crosses the subject crosswalk, then the subtraction of the yellow change interval and the red clearance interval in Equation 31-70 may cause some conflict between pedestrians and left-turning vehicles. If this conflict can occur, then the minimum green interval duration should be computed as $G_{p,min} = t_{pr} + (L_{cc}/S_p)$.

The second pedestrian consideration in timing design is the time required to serve pedestrian demand. The green interval duration should equal or exceed this time to ensure pedestrian demand is served each cycle. The time needed to serve this demand is computed with either Equation 31-71 or Equation 31-72, along with Equation 31-73.

If the crosswalk width W is greater than 10 ft, then

Equation 31-71

$$t_{ps} = 3.2 + \frac{L_{cc}}{S_p} + 2.7\frac{N_{ped}}{W}$$

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

Equation 31-72

$$t_{ps} = 3.2 + \frac{L_{cc}}{S_p} + 0.27\,N_{ped}$$

with

Equation 31-73

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

where

t_{ps} = pedestrian service time (s),

W = effective width of crosswalk (ft),

$v_{ped,i}$ = pedestrian flow rate in the subject crossing for travel direction i (p/h), and

N_{ped} = number of pedestrians crossing during an interval (p).

Equation 31-73 assumes pedestrians always cross at the start of the phase. Thus, it yields a conservatively large estimate of N_{ped} because some pedestrians arrive and cross during the green indication.

Equation 31-73 is specific to the pedestrian flow rate in one direction of travel along the subject crosswalk. If the pedestrian flow rate varies significantly during the analysis period for the crosswalk's two travel directions, then t_{ps} should be calculated for both travel directions, and the larger value should be used to estimate the green interval duration needed to serve pedestrian demand.

PEDESTRIAN AND BICYCLE ADJUSTMENT FACTORS

Exhibit 31-8 shows sample conflict zones where intersection users compete for space. This competition reduces the saturation flow rate of the turning vehicles. Its effect is quantified in the pedestrian and bicycle adjustment factors. This subsection describes a procedure for calculating these factors, which are used in the procedure for calculating the adjusted saturation flow rate that is described in Section 3 of Chapter 19.

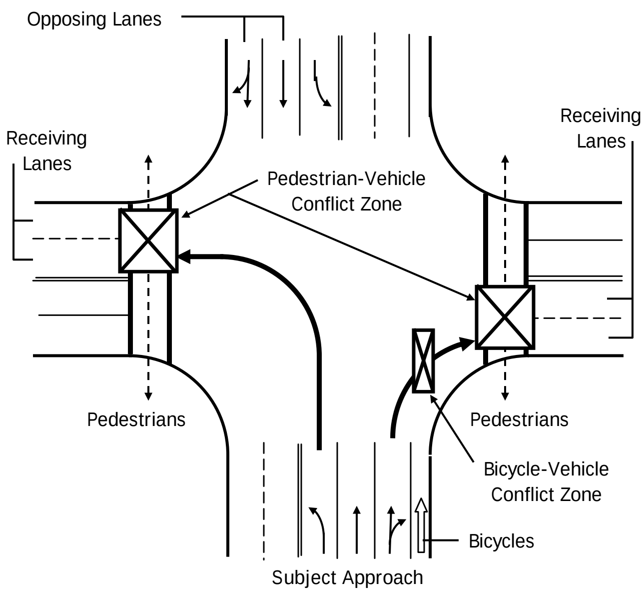


Exhibit 31-8
Conflict Zone Locations

This subsection consists of two subsections. The first subsection describes the procedure for computing (*a*) the pedestrian–bicycle adjustment factor for right-turn lane groups and (*b*) the pedestrian adjustment factor for left-turn lane groups from a one-way street. The second subsection describes the procedure for computing the pedestrian adjustment factor for left-turn groups served by permitted or protected-permitted operation.

The following guidance is used to determine the pedestrian adjustment factor for lane groups serving left-turn movements f_{Lpb}:

- If there are no conflicting pedestrians, then f_{Lpb} is equal to 1.0.
- If the lane group is on a two-way street and the protected mode or split phasing is used, then f_{Lpb} is equal to 1.0.
- If the lane group is on a one-way street, then the procedure described in the first subsection below is used to compute f_{Lpb}.
- If the lane group is on a two-way street and either the permitted mode or the protected-permitted mode is used, then the procedure described in the second subsection below is used to calculate f_{Lpb}.

The following guidance is used to determine the pedestrian–bicycle adjustment factor for lane groups serving right-turn movements f_{Rpb}:

- If there are no conflicting pedestrians or bicycles, then f_{Rpb} is equal to 1.0.
- If the protected mode is used, then f_{Rpb} is equal to 1.0.
- If the permitted mode or the protected-permitted mode is used, then the procedure described in the first subsection below is used to compute f_{Rpb}.

Right-Turn Movements and Left-Turn Movements from One-Way Street

A. Determine Pedestrian Flow Rate During Service

This procedure requires knowledge of the phase duration and cycle length. If these variables are not known and the intersection is pretimed, then they can be estimated by using the procedure described in the previous subsection titled Pretimed Phase Duration. If the intersection is actuated, then the average phase duration and cycle length can be computed by using the procedure described in the previous subsection titled Actuated Phase Duration.

The pedestrian flow rate during the pedestrian service time is computed with Equation 31-74.

Equation 31-74

$$v_{pedg} = v_{ped} \frac{C}{g_{ped}} \leq 5{,}000$$

where

v_{pedg} = pedestrian flow rate during the pedestrian service time (p/h),

v_{ped} = pedestrian flow rate in the subject crossing (walking in both directions) (p/h),

C = cycle length (s), and

g_{ped} = pedestrian service time (s).

If the phase providing service to pedestrians is actuated, has a pedestrian signal head, and rest-in-walk is not enabled, then the pedestrian service time is equal to the smaller of (*a*) the effective green time for the phase or (*b*) the sum of the walk and pedestrian clear settings [i.e., $g_{ped} = \min(g, \text{Walk} + PC)$]. Otherwise, the pedestrian service time can be assumed to equal the effective green time for the phase (i.e., $g_{ped} = g$).

B. Determine Average Pedestrian Occupancy

If the pedestrian flow rate during the pedestrian service time is 1,000 p/h or less, then the pedestrian occupancy is computed with Equation 31-75.

Equation 31-75

$$OCC_{pedg} = \frac{v_{pedg}}{2{,}000}$$

where OCC_{pedg} is the pedestrian occupancy.

If the pedestrian flow rate during the pedestrian service time exceeds 1,000 p/h, then Equation 31-76 is used.

Equation 31-76

$$OCC_{pedg} = 0.4 + \frac{v_{pedg}}{10{,}000} \leq 0.90$$

A practical upper limit on v_{pedg} of 5,000 p/h should be maintained when Equation 31-76 is used.

C. Determine Bicycle Flow Rate During Green

The bicycle flow rate during the green indication is computed with Equation 31-77.

Equation 31-77

$$v_{bicg} = v_{bic} \frac{C}{g} \leq 1{,}900$$

where

v_{bicg} = bicycle flow rate during the green indication (bicycles/h),

v_{bic} = bicycle flow rate (bicycles/h),

C = cycle length (s), and

g = effective green time (s).

D. Determine Average Bicycle Occupancy

The average bicycle occupancy is computed with Equation 31-78.

$$OCC_{bicg} = 0.02 + \frac{v_{bicg}}{2{,}700}$$ Equation 31-78

where OCC_{bicg} is the bicycle occupancy, and v_{bicg} is the bicycle flow rate during the green indication (bicycles/h).

A practical upper limit on v_{bicg} of 1,900 bicycles/h should be maintained when Equation 31-78 is used.

E. Determine Relevant Conflict Zone Occupancy

Equation 31-79 is used for right-turn movements with no bicycle interference or for left-turn movements from a one-way street. This equation is based on the assumptions that (*a*) pedestrian crossing activity takes place during the time period associated with g_{ped}, and (*b*) no crossing occurs during the green time period $g - g_{ped}$, when this time period exists.

$$OCC_r = \frac{g_{ped}}{g} OCC_{pedg}$$ Equation 31-79

where OCC_r is the relevant conflict zone occupancy.

Alternatively, Equation 31-80 is used for right-turn movements with pedestrian and bicycle interference, with all variables as previously defined.

$$OCC_r = \left(\frac{g_{ped}}{g} OCC_{pedg}\right) + OCC_{bicg} - \left(\frac{g_{ped}}{g} OCC_{pedg} OCC_{bicg}\right)$$ Equation 31-80

F. Determine Unoccupied Time

If the number of cross-street receiving lanes is equal to the number of turn lanes, then turning vehicles will not be able to maneuver around pedestrians or bicycles. In this situation, the time the conflict zone is unoccupied is computed with Equation 31-81.

$$A_{pbT} = 1 - OCC_r$$ Equation 31-81

where A_{pbT} is the unoccupied time, and OCC_r is the relevant conflict zone occupancy.

Alternatively, if the number of cross-street receiving lanes exceeds the number of turn lanes, turning vehicles will more likely maneuver around pedestrians or bicycles. In this situation, the effect of pedestrians and bicycles on saturation flow is lower, and the time the conflict zone is unoccupied is computed with Equation 31-82.

$$A_{pbT} = 1 - 0.6\, OCC_r$$ Equation 31-82

Either Equation 31-81 or Equation 31-82 is used to compute A_{pbT}. The choice of which equation to use should be based on careful consideration of the number of turn lanes and the number of receiving lanes. At some intersections, drivers may consistently and deliberately make illegal turns from an exclusive through lane. At other intersections, proper turning cannot be executed because the receiving lane is blocked by double-parked vehicles. For these reasons, the number of turn lanes and receiving lanes should be determined from field observation.

G. Determine Saturation Flow Rate Adjustment Factor

For permitted right-turn operation in an exclusive lane, Equation 31-83 is used to compute the pedestrian–bicycle adjustment factor.

Equation 31-83

$$f_{Rpb} = A_{pbT}$$

where f_{Rpb} is the pedestrian–bicycle adjustment factor for right-turn groups, and A_{pbT} is the unoccupied time.

For protected-permitted operation in an exclusive lane, the factor from Equation 31-83 is used to compute the adjusted saturation flow rate during the permitted period. The factor has a value of 1.0 when used to compute the adjusted saturation flow rate for the protected period.

For left-turn movements from a one-way street, Equation 31-84 is used to compute the pedestrian adjustment factor.

Equation 31-84

$$f_{Lpb} = A_{pbT}$$

where f_{Lpb} is the pedestrian adjustment factor for left-turn groups, and A_{pbT} is the unoccupied time.

Permitted and Protected-Permitted Left-Turn Movements

This subsection describes a procedure for computing the adjustment factor for left-turn movements on a two-way street that are operating in either the permitted mode or the protected-permitted mode. The calculations in this subsection supplement the procedure described in the previous subsection. The calculations described in Steps A and B in the previous subsection must be completed first (substitute the effective permitted green time g_p for g in Step A), after which the calculations described in this subsection are completed.

This procedure does not account for vehicle–bicycle conflict during the left-turn maneuver.

A. Compute Pedestrian Occupancy After Queue Clears

The pedestrian occupancy after the opposing queue clears is computed with Equation 31-85 or Equation 31-86. The opposing-queue service time g_q is computed as the effective permitted green time g_p less the duration of permitted left-turn green time that is not blocked by an opposing queue g_u (i.e., $g_q = g_p - g_u$).

If $g_q < g_{ped}$, then

Equation 31-85

$$OCC_{pedu} = OCC_{pedg}(1 - \frac{0.5\, g_q}{g_{ped}})$$

otherwise

$$OCC_{pedu} = 0.0$$

Equation 31-86

where OCC_{pedu} is the pedestrian occupancy after the opposing queue clears, g_q is the opposing-queue service time (= g_s for the opposing movement) (s), and all other variables are as previously defined.

If the opposing-queue service time g_q equals or exceeds the pedestrian service time g_{ped}, then the opposing queue consumes the entire pedestrian service time.

B. Determine Relevant Conflict Zone Occupancy

After the opposing queue clears, left-turning vehicles complete their maneuvers on the basis of accepted gap availability in the opposing traffic stream. Relevant conflict zone occupancy is a function of the probability of accepted gap availability and pedestrian occupancy. It is computed with Equation 31-87.

$$OCC_r = \frac{g_{ped} - g_q}{g_p - g_q}(OCC_{pedu})\, e^{-5.00\, v_o/3{,}600}$$

Equation 31-87

where v_o is the opposing demand flow rate (veh/h), g_p is the effective green time for permitted left-turn operation (s), and all other variables are as previously defined.

The opposing demand flow rate v_o is determined to be one of two cases. In Case 1, v_o equals the sum of the opposing through and right-turn volumes. In Case 2, v_o equals the opposing through volume. Case 2 applies when there is a through movement on the opposing approach and one of the following conditions applies: (*a*) there is an exclusive right-turn lane on the opposing approach and the analyst optionally indicates that this lane does not influence the left-turn drivers' gap acceptance, or (*b*) there is no right-turn movement on the opposing approach. Case 1 applies whenever Case 2 does not apply.

When an exclusive right-turn lane exists on the opposing approach, the default condition is to assume this lane influences the subject left-turn drivers' gap acceptance. The determination that the exclusive right-turn lane does not influence gap acceptance should be based on knowledge of local driver behavior, traffic conditions, and intersection geometry.

C. Determine Unoccupied Time

Either Equation 31-81 or Equation 31-82 from the previous subsection (i.e., Step F above) is used to compute A_{pbT}. The choice of which equation to use should be based on a consideration of the number of left-turn lanes and the number of receiving lanes.

D. Determine Saturation Flow Rate Adjustment Factor

Equation 31-88 is used to compute the pedestrian adjustment factor f_{Lpb} from A_{pbT}, the unoccupied time.

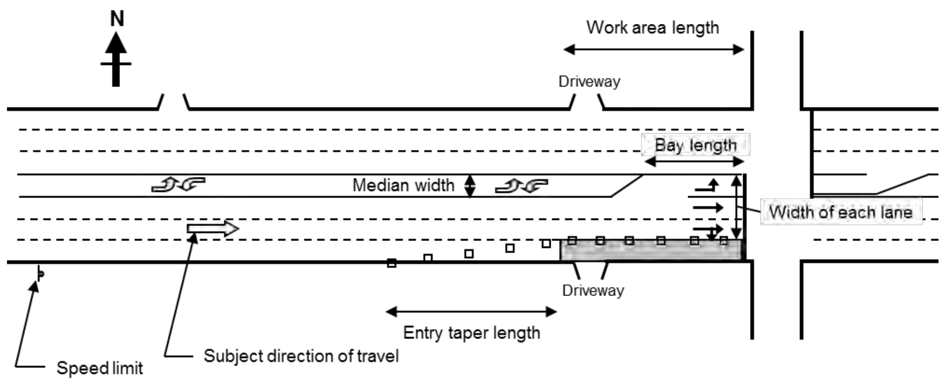
$$f_{Lpb} = A_{pbT}$$

Equation 31-88

WORK ZONE PRESENCE ADJUSTMENT FACTOR

The procedure described in this subsection can be used to evaluate signalized intersection operation when a work zone is present on the intersection approach. The work zone is considered to be on the intersection approach if some (or all) of the work zone is located between the stop line and a point 250 ft upstream of the stop line. The work zone may be located on the shoulder, or it may include the closure of one or more lanes. An intersection with a work zone located on the eastbound approach is shown in Exhibit 31-9.

Exhibit 31-9
Work Zone on an Intersection Approach



Required Input Data

The input data that are needed to estimate the effect of work zone presence on saturation flow rate are listed in Exhibit 31-10. The two data elements listed are described in this subsection. The contents of Exhibit 31-10 are in addition to those listed in Exhibit 19-11.

Exhibit 31-10
Geometric Design Input Data Requirements for Work Zones

Input Data Element and Units	Basis
Number of lanes open on the approach in the work zone (ln)	Approach
Approach lane width during work zone (ft)	Approach

Note: Approach = one value or condition for the intersection approach.

Number of Lanes Open on the Approach in the Work Zone

The number of lanes open on the approach in the work zone represents the count of left-turn and through lanes that are open during work zone presence. The count does not include any exclusive right-turn lanes that may exist. The count is taken in the work zone (not upstream or downstream of the work zone). If the number of lanes in the work zone varies, then the smallest number of lanes provided to motorists is used for this input variable.

Approach Lane Width During Work Zone

The approach lane width represents the total width of all open left-turn, through, and right-turn lanes on the intersection approach when the work zone is present.

Computational Steps

The saturation flow rate adjustment factor for the case in which a work zone is located at the intersection can be computed by using Equation 31-89 with Equation 31-90 and Equation 31-91.

$$f_{wz} = 0.858 \times f_{wid} \times f_{\text{reduce}} \leq 1.0$$

Equation 31-89

with

$$f_{wid} = \frac{1}{1 - 0.0057\,(a_w - 12)}$$

Equation 31-90

$$f_{\text{reduce}} = \frac{1}{1 + 0.0402\,(n_o - n_{wz})}$$

Equation 31-91

where

f_{wz} = adjustment factor for work zone presence at the intersection,

f_{wid} = adjustment factor for approach width,

f_{reduce} = adjustment factor for reducing lanes during work zone presence,

a_w = approach lane width during work zone (= total width of all open left-turn, through, and right-turn lanes) (ft),

n_o = number of left-turn and through lanes open during normal operation (ln), and

n_{wz} = number of left-turn and through lanes open during work zone presence (ln).

This factor is computed during Step 4, Determine Adjusted Saturation Flow Rate, of the motorized vehicle methodology in Chapter 19, Signalized Intersections. One value is computed for (and is applicable to) all lane groups on the subject intersection approach.

3. QUEUE ACCUMULATION POLYGON

This section describes a procedure for using the queue accumulation polygon (QAP) to estimate delay. The section consists of three subsections. The first subsection provides a review of concepts related to the QAP. The second subsection describes a general procedure for developing the QAP, and the third subsection extends the general procedure to the evaluation of left-turn lane groups.

The discussion in this section describes basic principles for developing polygons for selected types of lane assignment, lane grouping, left-turn operation, and phase sequence. The analyst is referred to the computational engine for specific calculation details, especially as they relate to assignments, groupings, left-turn operations, and phase sequences not addressed in this section. This engine is described in Section 7.

CONCEPTS

The QAP is a graphic tool for describing the deterministic relationship between vehicle arrivals, departures, queue service time, and delay. The QAP defines the queue size for a traffic movement as a function of time during the cycle. The shape of the polygon is defined by the following factors: arrival flow rate during the effective red and green intervals, saturation flow rate associated with each movement in the lane group, signal indication status, left-turn operation mode, and phase sequence. Once constructed, the polygon can be used to compute the queue service time, capacity, and uniform delay for the corresponding lane group.

A QAP is shown in Exhibit 31-11. The variables shown in the exhibit are defined in the following list:

r = effective red time = $C - g$ (s),

g = effective green time (s),

C = cycle length (s),

g_s = queue service time = $Q_r/(s - q_g)$ (s),

g_e = green extension time (s),

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

v = demand flow rate (veh/h),

q_r = arrival flow rate during the effective red time = $(1 - P)\ q\ C/r$ (veh/s),

q_g = arrival flow rate during the effective green time = $P\ q\ C/g$ (veh/s),

Q_r = queue size at the end of the effective red time = $q_r\, r$ (veh),

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

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

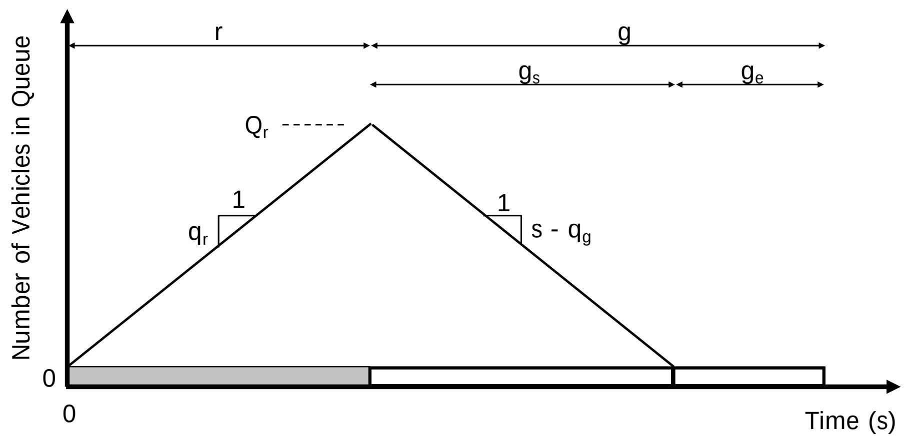


Exhibit 31-11
Queue Accumulation Polygon for Protected Movements

In application, all flow rate variables are converted to common units of vehicles per second per lane. The presentation in this section is based on these units for *q* and *s*.

The polygon in Exhibit 31-11 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. Other polygon shapes are possible, depending on whether the lane group includes a shared lane and whether the lane group serves a permitted (or protected-permitted) left-turn movement. In general, a unique polygon shape will be dictated by each combination of left-turn operational mode (i.e., permitted, protected, or protected-permitted) and phase sequence (i.e., lead, lag, or split). A general procedure for constructing these polygons is described in the next subsection.

GENERAL QAP CONSTRUCTION PROCEDURE

This subsection describes a general procedure for constructing a QAP for a lane group at a signalized intersection. It is directly applicable to left-turn lane groups that have exclusive lanes and protected operation, through lane groups with exclusive lanes, and right-turn lane groups with exclusive lanes. Variations that extend this procedure to turn lane groups with shared lanes, permitted operation, or protected-permitted operation are described in the next subsection.

The construction of a QAP is based on identification of flow rates and service times during the average signal cycle. These rates and times define periods of queue growth, queue service, and service upon arrival. As shown in Exhibit 31-11, the rates and times define queue size as it varies during the cycle. The resulting polygon formed by the queue size profile can be decomposed into a series of trapezoid or triangle shapes, with each shape having a known time interval. Collectively, the areas of the individual shapes can be added to equal the area of the polygon, and the time intervals can be added to equal the cycle length.

The QAP calculation sequence follows the order of interval occurrence over time, and the results can be recorded graphically (as in Exhibit 31-11) or in a tabular manner (i.e., row by row, where each row represents one time interval). A time interval is defined to begin and end at points when either the departure rate or the arrival rate changes. For the duration of the interval, these rates are assumed to be constant.

The following text outlines the calculation sequence used to construct a QAP for a specified lane group. The sequence is repeated for each lane group at the intersection, with the through lane groups evaluated first so the saturation flow rate of permitted left-turn lane groups can be based on the known queue service time for the opposing traffic movements.

1. The QAP calculations for a given lane group start with the end of the effective green period for the phase serving the subject lane group in a protected manner. The initial queue Q_i is assumed to equal 0.0 vehicles.
2. Determine the points in the cycle when the arrival flow rate or the discharge rate changes. The arrival rate may change because of platoons formed in response to an upstream signal, so it is expressed in terms of the arrival rate during green q_g and during red q_r. The discharge rate may change because of the start or end of effective green, a change in the saturation flow rate, the depletion of the subject queue, the depletion of the opposing queue, or the departure of left-turn vehicles as sneakers.
3. For the time interval between the points identified in Step 2, number each interval and compute its duration. Next, identify the arrival rate and discharge rate associated with the interval. Finally, confirm that the sum of all interval durations equals the cycle length.
4. Calculate the capacity of each interval for which there is some discharge, including sneakers when applicable. The sum of these capacities equals the total lane group capacity. Calculate the demand volume for each interval for which there are some arrivals. The sum of these volumes equals the total lane group volume.
5. Calculate the volume-to-capacity ratio X for the lane group by dividing the lane group's total volume by its total capacity. If the volume-to-capacity ratio exceeds 1.0, then calculate the adjusted arrival flow rate q' for each interval by dividing the original flow rate q by X (i.e., $q' = q/X$).
6. Calculate the queue at the end of interval i with Equation 31-92.

Equation 31-92

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

   where Q_i is the queue size at the end of interval i (veh), $t_{d,i}$ is the duration of time interval i during which the arrival flow rate and saturation flow rate are constant (s), and all other variables are as previously defined.

7. If the queue at the end of interval i equals 0.0 vehicles, then compute the duration of the trapezoid or triangle with Equation 31-93. The subject interval should be divided into two intervals, with the first interval having a duration of $t_{t,i}$ and the second interval having a duration of $t_{d,i} - t_{t,i}$. The second interval has starting and ending queues equal to 0.0 vehicles.

Equation 31-93

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

   where $t_{t,i}$ is the duration of trapezoid or triangle in interval i (s), w_q is the queue change rate (= discharge rate minus arrival rate) (veh/s), and all other variables are as previously defined.

8. Steps 6 and 7 are repeated for each interval in the cycle.
9. When all intervals are completed, the assumption of a zero starting queue (made in Step 1) is checked. The queue size computed for the last interval should always equal the initially assumed value. If this is not the case, then Steps 6 through 8 are repeated by using the ending queue size of the last interval as the starting queue size for the first interval.
10. When all intervals have been evaluated and the starting and ending queue sizes are equal, then the uniform delay can be calculated. This calculation starts with computing the area of each trapezoid or triangle. These areas are then added to determine the total delay. Finally, the total delay is divided by the number of arrivals per cycle to produce uniform delay. Equations for calculating uniform delay by using the QAP are described in Step 7 of the next subsection.

QAP CONSTRUCTION PROCEDURE FOR SELECTED LANE GROUPS

This subsection describes a seven-step procedure for constructing a QAP for selected lane groups. The focus is on left-turn movements in lane groups with shared lanes, permitted operation, or protected-permitted operation. However, there is some discussion of other lane groups, lane assignments, and operation. The procedure described in this subsection represents an extension of the general procedure described in the previous subsection.

Step 1. Determine Permitted Green Time

This step applies when the subject left-turn movement is served by using the permitted mode or the protected-permitted mode. Two effective green times are computed. One is the effective green time for permitted left-turn operation g_p. This green time occurs during the period when the adjacent and opposing through movements both have a circular green indication (after adjustment for lost time).

The other effective green time represents the duration of permitted left-turn green time that is not blocked by an opposing queue g_u. This green time represents the time during the effective green time for permitted left-turn operation g_p that is not used to serve the opposing queue. This time is available to the subject left-turn movement to filter through the conflicting traffic stream.

Exhibit 31-12 provides equations for computing the unblocked permitted green time for left-turn Movement 1 (see Exhibit 19-1) when Dallas left-turn phasing is *not* used. Similar equations can be derived for the other left-turn movements or when Dallas phasing is used. The variables defined in this exhibit are provided in the following list:

g_u = duration of permitted left-turn green time that is not blocked by an opposing queue (s),

G_U = displayed green interval corresponding to g_u (s),

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

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

G_q = displayed green interval corresponding to g_q (s),

D_p = phase duration (s),

R_c = red clearance interval (s),

Y = yellow change interval (s), and

g_q = opposing-queue service time (= g_s for the opposing movement) (s).

Exhibit 31-12
Unblocked Permitted Green Time

Phase Sequence (phase numbers shown in boxes)		Displayed Unblocked Permitted Green Time G_U (s)[a]	Permitted Start-Up Lost Time $l_{1,p}$ (s)[b]	Permitted Extension Time e_p (s)[c]
Lead–Lead	[1][2] / [5][6]	$G_{U1} = \min[D_{p1} + D_{p2} - D_{p5} - Y_6 - R_{c6},\ G_{U1}^*]$ with $G_{U1}^* = D_{p2} - Y_6 - R_{c6} - G_{q2}$	$l_{1,1}^*$	e_1
	[1][2] / [5][6]	$G_{U1} = D_{p2} - Y_6 - R_{c6} - G_{q2}$	$l_{1,1}^*$	e_1
Lead–Lag or Lead–Perm	[1][2] / [6][5]	$G_{U1} = D_{p6} - Y_6 - R_{c6} - D_{p1} - G_{q2}$	0.0	e_1
	[1][2] / [6][5]	No permitted period	Not applicable	Not applicable
	[1][2] / [6]	$G_{U1} = D_{p6} - Y_6 - R_{c6} - D_{p1} - G_{q2}$	0.0	e_1
Lag–Lead or Lag–Perm	[2][1] / [5][6]	No permitted period	Not applicable	Not applicable
	[2][1] / [5][6]	$G_{U1} = D_{p2} - Y_2 - R_{c2} - \max[D_{p5},\ G_{q2}]$	$l_{1,1}$	0.0
	[2][1] / [6]	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2},\ D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	0.0
Perm–Lead	[2] / [5][6]	$G_{U1} = D_{p2} - Y_2 - R_{c2} - \max[D_{p5},\ G_{q2}]$	$l_{1,1}$	e_1
Perm–Lag	[2] / [6][5]	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2},\ D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	e_1
Perm–Perm	[2] / [6]	$G_{U1} = D_{p2} - Y_6 - R_{c6} - G_{q2}$	$l_{1,1}$	e_1
Lag–Lag	[2][1] / [6][5]	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2},\ D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	e_1^*
	[2][1] / [6][5]	$G_{U1} = \min[D_{p2} - Y_2 - R_{c2},\ D_{p6} - Y_6 - R_{c6}] - G_{q2}$	$l_{1,1}$	e_1^*

Notes: [a] G_{q2} is computed for each opposing lane (excluding any opposing shared left-turn lane), and the value used corresponds to the lane requiring the longest time to clear. In general, if the opposing lanes serve through movements exclusively, then $G_{q2} = g_q + l_1$. If an opposing lane is shared, then $G_{q2} = g_p - g_e + l_1$, where g_p is the effective green time for permitted operation (s), g_e is the green extension time (s), and l_1 is the start-up lost time (s).
[b] If $D_{p5} > (D_{p1} - Y_1 - R_{c1})$, then $l_1^* = D_{p5} - (D_{p1} - Y_1 - R_{c1}) + l_1 - e_1$; otherwise, $l_1^* = 0.0$. Regardless, the result should not be less than 0.0 or more than l_1.
[c] $e_1^* = D_{p2} - (D_{p6} - Y_6 - R_{c6})$, provided the result is not less than 0.0 or more than e_1.
Perm = permitted.

For the first four variables in the preceding list, the subscript "1" is added to the variable when it is used in an Exhibit 31-12 equation. This subscript denotes Movement 1. For the next four variables in the list, a numeric subscript is added to the variable when it is used in an equation from the exhibit. This subscript denotes the phase number associated with the variable. Exhibit 31-12 applies only to left-turn Movement 1. The subscripts need to be changed to apply the equations to other left-turn movements.

The equations shown in Exhibit 31-12 indicate that the effective green time for the permitted operation of Phase 1 depends on the duration of Phase 2 and sometimes the duration of Phase 5. In all instances, Movement 1 has permitted operation during all, or a portion of, Phase 6.

For a given left-turn lane group, one of the equations in the second column (Displayed Unblocked Permitted Green Time) of Exhibit 31-12 will apply. It is used to compute the displayed green interval corresponding to g_u (i.e., G_U). The computed G_U is required to have a nonnegative value. If the calculation yields a negative value, then G_U is set to 0.0.

The same equation can be used to compute the displayed green interval corresponding to g_p (i.e., G_p) by substituting G_p for G_U and 0.0 for G_q. Again, the computed G_p is required to have a nonnegative value. If the calculation yields a negative value, then G_p is set to 0.0.

Equation 31-94 is used to compute the effective green time for permitted left-turn operation.

$$g_p = G_p - l_{1,p} + e_p \geq 0.0$$

Equation 31-94

where

g_p = effective green time for permitted left-turn operation (s),

G_p = displayed green interval corresponding to g_p (s),

$l_{1,p}$ = permitted start-up lost time (s), and

e_p = permitted extension of effective green (s).

The values of $l_{1,p}$ and e_p used in Equation 31-94 are obtained from the two right-hand columns (Permitted Start-Up Lost Time and Permitted Extension Time, respectively) of Exhibit 31-12.

The start-up lost time for g_u is considered to occur coincident with the start-up lost time associated with g_p. Hence, if the opposing-queue service time consumes an initial portion of g_p, then there is no start-up lost time associated with g_u. The rationale for this approach is that left-turn drivers waiting for the opposing queue to clear will be anticipating queue clearance and may be moving forward slowly (perhaps already beyond the stop line) so that there is negligible start-up lost time at this point. This approach also accommodates the consideration of multiple effective green-time terms when there is a shared lane (e.g., g_f), and it avoids inclusion of multiple start-up lost times during g_p. In accordance with this rationale, Equation 31-95 is used to compute the permitted left-turn green time that is not blocked by an opposing queue g_u, where all other variables are as previously defined.

$$g_u = G_u + e_p \leq g_p$$

Equation 31-95

If protected-permitted operation exists and Dallas phasing is used, then the displayed green interval corresponding to g_u (i.e., G_U) is equal to the opposing through phase duration minus the queue service time and change period of the opposing through phase (i.e., $G_{U1} = D_{p2} - Y_2 - R_{c2} - G_{q2}$). The permitted start-up lost time $l_{1,p}$ and permitted extension of effective green e_p are equal to l_1 and e, respectively. Otherwise, all the calculations described previously apply.

Step 2. Determine Time Before First Left-Turn Vehicle Arrives

This step applies when the left-turn movement is served by using the permitted mode on a shared-lane approach. The variable of interest represents the time that elapses from the start of the permitted green to the arrival of the first left-turning vehicle at the stop line. During this time, through vehicles in the shared lane are served at the saturation flow rate of an exclusive through lane.

Considerations of vehicle distribution impose an upper limit on the time before the first left-turn vehicle arrives when it is used to define a period of saturation flow. This limit is computed with Equation 31-96.

Equation 31-96

$$g_{f,max} = \frac{(1 - P_L)}{0.5\, P_L}\left(1 - [1 - P_L]^{0.5\, g_p}\right) - l_{1,p} \geq 0.0$$

where $g_{f,max}$ is the maximum time before the first left-turning vehicle arrives and within which there are sufficient through vehicles to depart at saturation (s), P_L is the proportion of left-turning vehicles in the shared lane (decimal), and all other variables are as previously defined.

The value of 0.5 in two locations in Equation 31-96 represents the approximate saturation flow rate (in vehicles per second) of through vehicles in an exclusive lane. This approximation simplifies the calculation and provides sufficient accuracy in the estimate of $g_{f,max}$.

The time before the first left-turning vehicle arrives and blocks the shared lane is computed with Equation 31-97 or Equation 31-98, along with Equation 31-99.

If the approach has one lane, then

Equation 31-97

$$g_f = \max\left(G_p\, e^{-0.860\, LTC^{0.629}} - l_{1,p}, 0.0\right) \leq g_{f,max}$$

otherwise

Equation 31-98

$$g_f = \max\left(G_p\, e^{-0.882\, LTC^{0.717}} - l_{1,p}, 0.0\right) \leq g_{f,max}$$

with

Equation 31-99

$$LTC = \frac{v_{lt}\, C}{3{,}600}$$

where

g_f = time before the first left-turning vehicle arrives and blocks the shared lane (s),

LTC = left-turn flow rate per cycle (veh/cycle), and

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

The approach is considered to have one lane for this step if (*a*) there is one lane serving all vehicles on the approach and (*b*) the left-turn movement on this approach shares the one lane.

Step 3. Determine Permitted Left-Turn Saturation Flow Rate

This step applies when left-turning vehicles are served by using the permitted mode or the protected-permitted mode from an exclusive lane. The saturation flow rate for permitted left-turn operation is calculated with Equation 31-100.

$$s_p = \frac{v_o \, e^{-v_o \, t_{cg}/3{,}600}}{1 - e^{-v_o \, t_{fh}/3{,}600}}$$

Equation 31-100

where

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln),

v_o = opposing demand flow rate (veh/h),

t_{cg} = critical headway = 4.5 (s), and

t_{fh} = follow-up headway = 2.5 (s).

The opposing demand flow rate v_o is determined to be one of two cases. In Case 1, v_o equals the sum of the opposing through and right-turn volumes. In Case 2, v_o equals the opposing through volume. Case 2 applies when there is a through movement on the opposing approach and one of the following conditions applies: (*a*) there is an exclusive right-turn lane on the opposing approach and the analyst optionally indicates that this lane does not influence the left-turn drivers' gap acceptance, or (*b*) there is no right-turn movement on the opposing approach. Case 1 applies whenever Case 2 does not apply.

When an exclusive right-turn lane exists on the opposing approach, the default condition is to assume this lane influences the subject left-turn drivers' gap acceptance. The determination that the exclusive right-turn lane does not influence gap acceptance should be based on knowledge of local driver behavior, traffic conditions, and intersection geometry.

In those instances in which the opposing volume equals 0.0 veh/h during the analysis period, the opposing volume is set to a value of 0.1 veh/h.

The opposing demand flow rate is not adjusted for unequal lane use in this equation. Increasing this flow rate to account for unequal lane use would misrepresent the frequency and size of headways in the opposing traffic stream. Thus, this adjustment would result in the left-turn saturation flow rate being underestimated.

Step 4. Determine Through-Car Equivalent

This step applies when left-turning vehicles are served by using the permitted mode or the protected-permitted mode. Two variables are computed to quantify the relationship between left-turn saturation flow rate and the base saturation flow rate. The first variable represents the more common case in which left-turning vehicles filter through an oncoming traffic stream. It is computed from Equation 31-101.

$$E_{L1} = \frac{s_o}{s_p}$$

Equation 31-101

where

E_{L1} = equivalent number of through cars for a permitted left-turning vehicle,

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

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln).

The second variable to be computed represents the case in which the opposing approach has one lane. It describes the saturation flow rate during the time interval coincident with the queue service time of the opposing queue. For this case, the saturation flow rate during the period after the arrival of the first blocking left-turning vehicle and before the end of the opposing-queue service time is influenced by the proportion of left-turning vehicles in the opposing traffic stream. These vehicles create artificial gaps in the opposing traffic stream through which the blocking left-turning vehicles on the subject approach can turn. This effect is considered through calculation of the following through-car equivalency factor by using Equation 31-102 with Equation 31-103.

Equation 31-102

$$E_{L2} = \frac{1 - (1 - P_{lto})^{n_q}}{P_{lto}} \geq E_L$$

with

Equation 31-103

$$n_q = 0.278(g_p - g_u - g_f) \geq 0.0$$

where

E_{L2} = equivalent number of through cars for a permitted left-turning vehicle when opposed by a queue on a single-lane approach,

P_{lto} = proportion of left-turning vehicles in the opposing traffic stream (decimal),

n_q = maximum number of opposing vehicles that could arrive after g_f and before g_u (veh), and

all other variables are as previously defined.

The value of 0.278 in Equation 31-103 represents the approximate saturation flow rate (in vehicles per second) of vehicles in the opposing shared lane. This approximation simplifies the calculation and provides sufficient accuracy in the estimation of n_q.

There is one lane on the opposing approach when this approach has one lane serving through vehicles, a left-turn movement that shares the through lane, and one of the following conditions applies: (*a*) there is an exclusive right-turn lane on the opposing approach and the analyst optionally indicates that this lane does not influence the left-turn drivers' gap acceptance, (*b*) there is a right-turn movement on the opposing approach and it shares the through lane, or (*c*) there is no right-turn movement on the opposing approach.

When an exclusive right-turn lane exists on the opposing approach, the default condition is to assume this lane influences the subject left-turn drivers' gap acceptance. The determination that the exclusive right-turn lane does not influence gap acceptance should be based on knowledge of local driver behavior, traffic conditions, and intersection geometry.

Step 5. Determine Proportion of Turns in a Shared Lane

This step applies when turning vehicles share a lane with through vehicles and the approach has two or more lanes. The proportion of turning vehicles in the shared lane is used in the next step to determine the saturation flow rate for the shared lane.

The proportion of left-turning vehicles in the shared lane P_L is computed if the shared lane includes left-turning vehicles. The proportion of right-turning vehicles in the shared lane P_R is computed if the shared lane includes right-turning vehicles. Guidance for computing these two variables is provided in Section 2.

If the approach has one traffic lane, then P_L equals the proportion of left-turning vehicles on the subject approach P_{lt}, and P_R equals the proportion of right-turning vehicles on the subject approach P_{rt}.

Step 6. Determine Lane Group Saturation Flow Rate

The saturation flow rate for the lane group is computed during this step. When the lane group consists of an exclusive lane operating in the protected mode, then it has one saturation flow rate. This rate equals the adjusted saturation flow rate computed by the procedure described in the motorized vehicle methodology in Section 3 of Chapter 19.

The focus of discussion in this step is the calculation of saturation flow rate for lane groups that are *not* in an exclusive lane or operating in the protected mode. Thus, the discussion in this step focuses on shared-lane lane groups and lane groups for which the permitted or protected-permitted mode is used. As the discussion indicates, these lane groups often have two or more saturation flow rates, depending on the phase sequence and operational mode of the turn movements.

Permitted Right-Turn Operation in Exclusive Lane

The saturation flow rate for a permitted right-turn operation in an exclusive lane is computed with Equation 31-104.

$$s_r = s_o\, f_w\, f_{HVg}\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{RT}\, f_{Rpb}\, f_{wz}\, f_{ms}\, f_{sp}$$

Equation 31-104

where s_r is the saturation flow rate in an exclusive right-turn lane group with permitted operation (veh/h/ln), and the other variables are defined following Equation 19-8 in Chapter 19.

Permitted Right-Turn Operation in Shared Lane

The saturation flow rate for permitted right-turn operation in a shared lane is computed with Equation 31-105.

$$s_{sr} = \frac{s_{th}}{1 + P_R\left(\dfrac{E_R}{f_{Rpb}} - 1\right)}$$

Equation 31-105

where

s_{sr} = saturation flow rate in shared right-turn and through lane group with permitted operation (veh/h/ln),

s_{th} = saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, area type, work zone presence, downstream lane blockage, and spillback) (veh/h/ln),

P_R = proportion of right-turning vehicles in the shared lane (decimal),

E_R = equivalent number of through cars for a protected right-turning vehicle = 1.18, and

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

The value of f_{Rpb} is obtained by the procedure described in Section 2.

Protected-Permitted Right-Turn Operation in Exclusive Lane

Two saturation flow rates are associated with protected-permitted operation. The saturation flow rate during the protected period s_{rt} is computed with Equation 31-106.

Equation 31-106

$$s_{rt} = s_o\, f_w\, f_{HVg}\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{RT}\, f_{wz}\, f_{ms}\, f_{sp}$$

where s_{rt} is the saturation flow rate of an exclusive right-turn lane with protected operation (veh/h/ln), and the other variables are defined following Equation 19-8 in Chapter 19.

The saturation flow rate during the permitted period is computed with Equation 31-104.

Permitted Left-Turn Operation in Shared Lane

There are three possible saturation flow periods during the effective green time associated with permitted left-turn operation in a shared lane. The first period occurs before the arrival of the first left-turning vehicle in the shared lane. This left-turning vehicle will block the shared lane until the opposing queue clears and a gap is available in the opposing traffic stream. The duration of this flow period is g_f. The saturation flow during this period is equal to s_{th}.

The second period of flow begins after g_f and ends with clearance of the opposing queue. It is computed with Equation 31-107.

Equation 31-107

$$g_{diff} = g_p - g_u - g_f \geq 0.0$$

where g_{diff} is the supplemental service time (s), and all other variables are as previously defined. This period may or may not exist, depending on the values of g_u and g_f.

If there are two or more opposing traffic lanes, then the saturation flow during the second period s_{sl2} equals 0.0 veh/h/ln. However, if the opposing approach has only one traffic lane, then the flow during this period occurs at a reduced rate that reflects the blocking effect of left-turning vehicles as they await an opposing left-turning vehicle. Left-turning vehicles during this period are assigned a through-car equivalent E_{L2}. The saturation flow rate for the shared lane is computed with Equation 31-108.

Equation 31-108

$$s_{sl2} = \frac{s_{th}}{1 + P_L\left(\frac{E_{L2}}{f_{Lpb}} - 1\right)}$$

where s_{sl2} is the saturation flow rate in the shared left-turn and through lane group during Period 2 (veh/h/ln), P_L is the proportion of left-turning vehicles in the shared lane (decimal), and all other variables are as previously defined.

There is one lane on the opposing approach when this approach has one lane serving through vehicles, a left-turn movement that shares the through lane, and one of the following conditions applies: (*a*) there is an exclusive right-turn lane on the opposing approach and the analyst optionally indicates that this lane does not influence the left-turn drivers' gap acceptance, (*b*) there is a right-turn movement on the opposing approach and it shares the through lane, or (*c*) there is no right-turn movement on the opposing approach.

When an exclusive right-turn lane exists on the opposing approach, the default condition is to assume this lane influences the subject left-turn drivers' gap acceptance. The determination that the exclusive right-turn lane does not influence gap acceptance should be based on knowledge of local driver behavior, traffic conditions, and intersection geometry.

The third period of flow begins after clearance of the opposing queue or arrival of the first blocking left-turn vehicle, whichever occurs last. Its duration equals the smaller of $g_p - g_f$ or g_u. The saturation flow rate for this period is computed with Equation 31-109.

Equation 31-109

$$s_{sl3} = \frac{s_{th}}{1 + P_L\left(\frac{E_{L1}}{f_{Lpb}} - 1\right)}$$

where s_{sl3} is the saturation flow rate in the shared left-turn and through lane group during Period 3 (veh/h/ln).

For multiple-lane approaches, the impact of the shared lane is extended to include the adjacent through traffic lanes. Specifically, queued drivers are observed to maneuver from lane to lane on the approach to avoid delay associated with the left-turning vehicles in the shared lane. The effect of this impact is accounted for by multiplying the saturation flow rate of the adjacent lanes by a factor of 0.91.

Permitted Left-Turn Operation in Exclusive Lane

There are two possible saturation flow periods during the effective green time associated with permitted left-turn operation in an exclusive lane. The two flow periods are discussed in reverse order, with the second period of flow discussed first.

The second period of flow begins after clearance of the opposing queue. Its duration is g_u. The saturation flow rate for this period is computed with Equation 31-110.

Equation 31-110

$$s_l = s_p\, f_w\, f_{HVg}\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{Lpb}\, f_{wz}\, f_{ms}\, f_{sp}$$

where s_l is the saturation flow rate in an exclusive left-turn lane group with permitted operation (veh/h/ln), and all other variables are defined following Equation 19-8 in Chapter 19.

The first period of flow begins with the start of the effective green period and ends with the clearance of the opposing queue. It is computed by using Equation 31-107 with the variable g_f equal to 0.0.

If there are two or more opposing traffic lanes, then the saturation flow during the first period s_{l1} equals 0.0 veh/h/ln. However, if the opposing approach has only one traffic lane, then the saturation flow rate is computed with Equation 31-111.

Equation 31-111

$$s_{l1} = \frac{s_l}{\left(\frac{E_{L2}}{f_{Lpb}}\right)}$$

where s_{l1} is the saturation flow rate in the exclusive left-turn lane group during Period 1 (veh/h/ln). The discussion following Equation 31-108 provides guidance for determining whether the opposing approach has only one traffic lane.

Protected-Permitted Left-Turn Operation in Exclusive Lane

Two saturation flow rates are associated with protected-permitted operation. The saturation flow rate during the protected period s_{lt} is computed with Equation 31-112.

Equation 31-112

$$s_{lt} = s_o\, f_w\, f_{HVg}\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{wz}\, f_{ms}\, f_{sp}$$

where s_{lt} is the saturation flow rate of an exclusive left-turn lane with protected operation (veh/h/ln), and all other variables are defined following Equation 19-8 in Chapter 19.

The saturation flow rate during the permitted period is computed with Equation 31-110. The duration of the permitted period is equal to g_u.

Protected-Permitted Left-Turn Operation in Shared Lane

The use of a protected-permitted operation in a shared lane has some special requirements to ensure safe and efficient operation. This operational mode requires display of the green ball when the left-turn green arrow is displayed (i.e., the green arrow is not displayed without also displaying the circular green). The following conditions are applied for actuated, protected-permitted operation in a shared lane:

- The left-turn phase is set to minimum recall.
- The maximum green setting for the left-turn phase must be less than or equal to the minimum green for the adjacent through phase.
- If both opposing approaches have protected-permitted operation in a shared lane, then the phase sequence must be lead–lag.
- No vehicle detection is assigned to the left-turn phase.
- Vehicle detection in the shared lane is assigned to the adjacent through movement phase.

There are four possible saturation flow periods during the effective green time associated with protected-permitted left-turn operation in a shared lane. The first three periods are the same as those for permitted left-turn operation in a shared lane (as described above).

The fourth period of flow coincides with the left-turn phase (i.e., the protected period). Its duration is equal to the effective green time for the left-turn phase g_l. The flow rate during this period is computed with Equation 31-113.

$$s_{sl4} = \frac{s_{th}}{1 + P_L(E_L - 1)}$$

Equation 31-113

where s_{sl4} is the saturation flow rate in the shared left-turn and through lane group during Period 4 (veh/h/ln).

For multiple-lane approaches, the impact of the shared lane is extended to include the adjacent through lanes. This impact is accounted for by multiplying the saturation flow rate of the adjacent lanes by a factor of 0.91.

Protected Left- and Right-Turn Operation in a Shared Lane

The saturation flow rate in a shared left- and right-turn lane group with protected operation is computed with Equation 31-114.

$$s_{lr} = \frac{s_{th}}{1 + P_L(E_L - 1) + P_R(E_R - 1)}$$

Equation 31-114

where s_{lr} is the saturation flow rate in the shared left- and right-turn lane group (veh/h/ln).

Step 7. Define Queue Accumulation Polygon

During this step, the green times and saturation flow rates are used to construct the QAP associated with each lane group. The polygon is then used to estimate uniform delay and queue service time. The lane group with the longest queue service time dictates the queue service time for the phase.

The QAP in Exhibit 31-11 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. This polygon also applies to split phasing and to shared lane groups serving through and right-turning vehicles operating with the permitted mode. For split phasing, each approach is evaluated separately to determine its queue service time and uniform delay. If the approach has left- or right-turn lanes, then a separate polygon is constructed for each turn lane group.

More complicated combinations of lane assignment, phase sequence, and left-turn operational mode dictate more complicated polygons. A polygon (or its tabular equivalent) must be derived for each combination. The most common combinations are illustrated in Exhibit 31-13 through Exhibit 31-16.

Exhibit 31-13
QAP for Permitted Left-Turn Operation in an Exclusive Lane

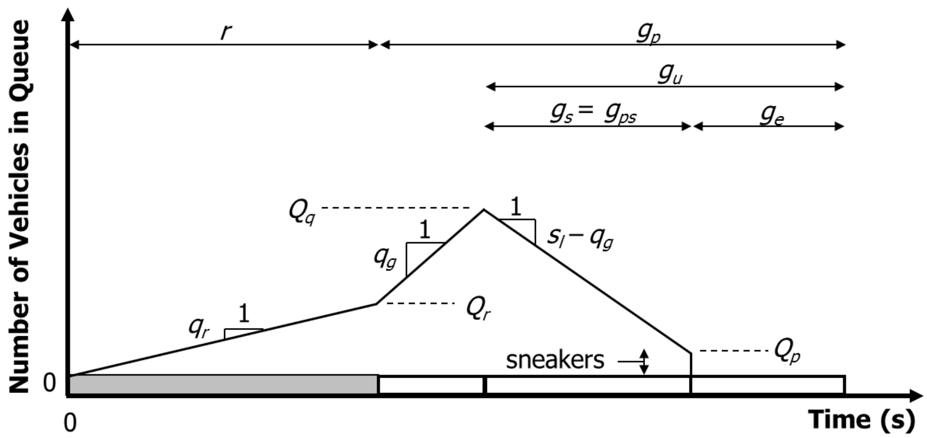


Exhibit 31-14
QAP for Permitted Left-Turn Operation in a Shared Lane

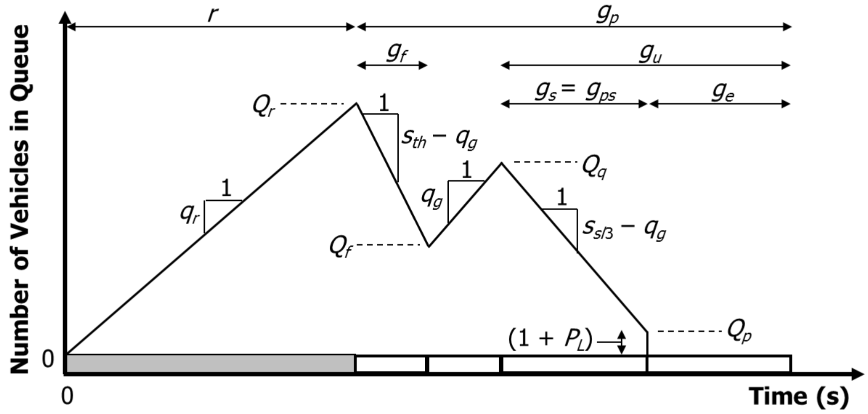
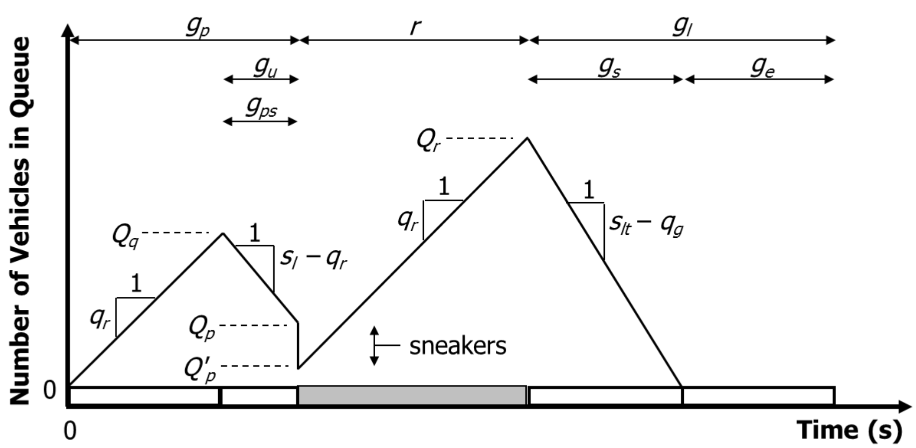


Exhibit 31-15
QAP for Leading, Protected-Permitted Left-Turn Operation in an Exclusive Lane



The concept is extended to shared left-turn and through lane groups with protected-permitted operation in Exhibit 31-17 and Exhibit 31-18. Other polygon shapes exist, depending on traffic flow rates, phase sequence, lane use, and left-turn operational mode. The concept of polygon construction must be extended to these other combinations to accurately estimate queue service time and uniform delay.

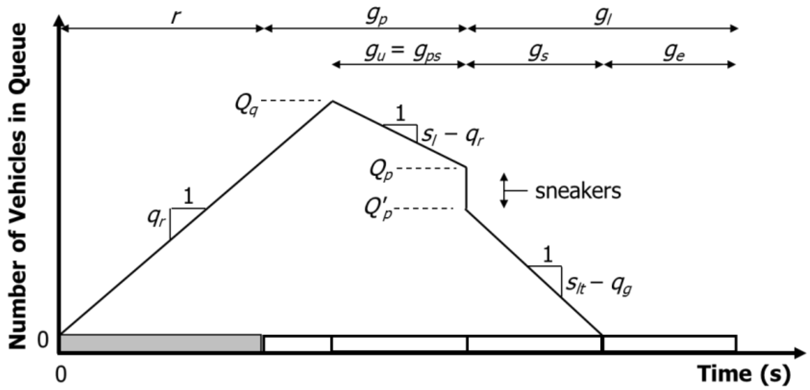


Exhibit 31-16
QAP for Lagging, Protected-Permitted Left-Turn Operation in an Exclusive Lane

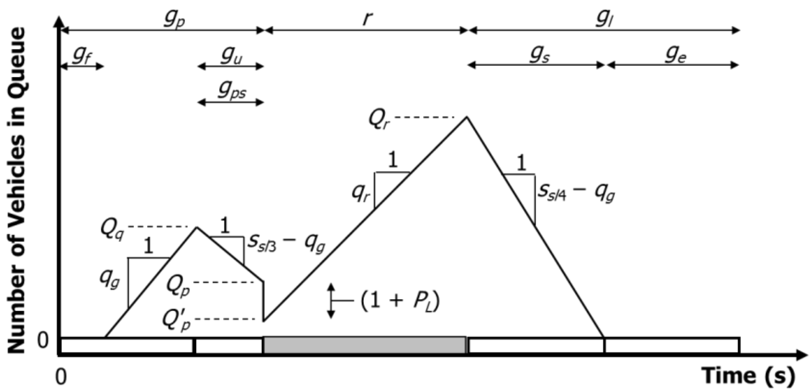


Exhibit 31-17
QAP for Leading, Protected-Permitted Left-Turn Operation in a Shared Lane

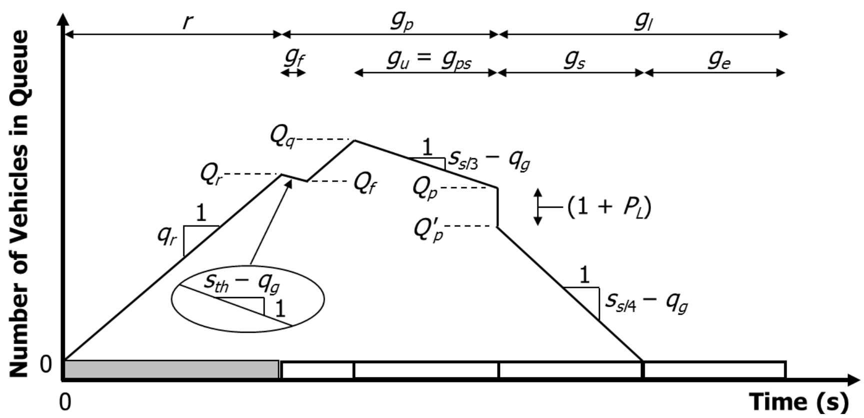


Exhibit 31-18
QAP for Lagging, Protected-Permitted Left-Turn Operation in a Shared Lane

Most of the variables shown in the following exhibits are defined in a previous subsection. Other variables are defined as follows:

g_l = effective green time for left-turn phase (s);

g_{ps} = queue service time during permitted left-turn operation (s);

Q_q = queue size at the start of g_u (veh);

Q_p = queue size at the end of permitted service time (veh);

Q'_p = queue size at the end of permitted service time, adjusted for sneakers (veh); and

Q_f = queue size at the end of g_f (veh).

The polygon in Exhibit 31-13 applies to the left-turn lane group with an exclusive lane that operates in the permitted mode during the adjacent through phase. If the phase extends to max-out, then some left-turning vehicles will be served as sneakers. The expected number of sneakers for this mode is reduced if downstream lane blockage or spillback is present [i.e., sneakers = $n_s f_{ms} f_{sp}$, where n_s is the number of sneakers per cycle = 2.0 (veh), f_{ms} is the adjustment factor for downstream lane blockage, and f_{sp} is the adjustment factor for sustained spillback].

The polygon in Exhibit 31-14 applies to the left-turn and through lane group on a shared lane approach with permitted operation. If the phase extends to max-out, then some left-turning vehicles will be served as sneakers. The expected number of sneakers (shown as $1 + P_L$) is computed as $(1 + P_L) f_{ms} f_{sp}$, where P_L is the proportion of left-turning vehicles in the shared lane.

The polygon in Exhibit 31-15 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and an exclusive lane. The polygon in Exhibit 31-16 applies to left-turn movements that have protected-permitted operation with a lagging left-turn phase and an exclusive lane. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the number of sneakers (where sneakers = $n_s f_{ms} f_{sp}$).

The polygon in Exhibit 31-17 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and a shared left-turn and through lane group. The polygon in Exhibit 31-18 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the expected number of sneakers [which is computed as $(1 + P_L) f_{ms} f_{sp}$].

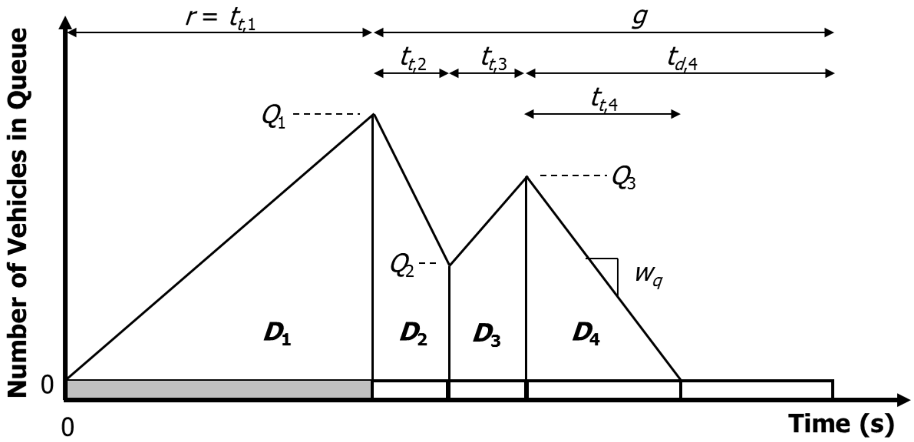
As noted above, all polygons are based on the requirement that lane volume cannot exceed lane capacity for the purpose of estimating the queue service time. This requirement is met in the polygons shown because the queue size equals 0.0 vehicles at some point during the cycle.

Exhibit 31-14 through Exhibit 31-18 are shown to indicate that queue size equals 0.0 vehicles at the start of the cycle (i.e., time = 0.0 s). In fact, the queue may not equal 0.0 vehicles at the start of the cycle for some signal timing and traffic conditions. Rather, there may be a nonzero queue at the start of the cycle, and a queue of 0.0 vehicles may not be reached until a different time in the cycle. Thus, in modeling any of the polygons in Exhibit 31-14 through Exhibit 31-18, an iterative process is required. For the first iteration, the queue is assumed to equal 0.0 vehicles at the start of the cycle. The polygon is then constructed, and the queue status is checked at the end of the cycle. If the queue at the end of the cycle is not 0.0 vehicles, then this value is used as a starting point in a second polygon construction. The second polygon will result in a queue at the end of the cycle that equals the queue used at the start of the cycle. Moreover, a queue value of 0.0 vehicles will occur at some point in the cycle.

A. Compute Uniform Delay and Queue Service Time

The procedure for calculating uniform delay and queue service time is described in this step. Exhibit 31-19 is used for this purpose.

Exhibit 31-19
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 31-115 with Equation 31-116.

$$d_1 = \frac{0.5 \sum_{i=1}(Q_{i-1} + Q_i)\ t_{t,i}}{q\ C}$$

Equation 31-115

with

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

Equation 31-116

where d_1 is the uniform delay (s/veh), $t_{t,i}$ is the duration of trapezoid or triangle in interval i (s), w_q is the queue change rate (i.e., slope of the upper boundary of the trapezoid or triangle) (veh/s), and all other variables are as previously defined.

The summation term in Equation 31-115 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 decrease to 0.0 vehicles and $t_{t,i}$ will be only as long as the time required for the queue to dissipate (= Q_{i-1}/w_q). This condition is shown to occur during Time Interval 4 in Exhibit 31-19.

The time required for the queue to dissipate represents the queue service time. The queue can dissipate during one or more intervals for turn movements that operate in the protected-permitted mode and for shared-lane lane groups.

For lane groups with exclusive lanes and protected operation, there is one queue service time. It is followed by the green extension time.

For permitted left-turn operation in an exclusive lane, there is one queue service time. It is followed by the green extension time.

For permitted left-turn operation in a shared lane, there can be two queue service times. The green extension time follows the last service time to occur.

For protected-permitted left-turn operation in an exclusive lane, there can be two queue service times. The service time that ends during the protected period is followed by the green extension time.

For protected-permitted left-turn operation in a shared lane, there can be three queue service times. The green extension time can follow the service time that ends during the protected period, but it is more likely to follow the last service time to occur during the permitted period.

For phases serving through or right-turning vehicles in two or more lane groups, the queue service time is measured from the start of the phase to the time when the queue in each lane group has been serviced (i.e., the longest queue service time controls). This consideration is extended to lane groups with shared through and left-turning vehicles.

B. Calculate Lane Group Capacity

This step describes the procedure used to calculate lane group capacity. It is based on the QAP and considers all opportunities for service during the cycle. The equations vary, depending on the left-turn operational mode, phase sequence, and lane assignments for the subject lane group.

Protected Left-Turn Operation in Exclusive Lane

The capacity for a protected left-turn operation in an exclusive-lane lane group is computed with Equation 31-117.

Equation 31-117

$$c_{l,e,p} = \frac{g_l \, s_{lt}}{C} N_l$$

where $c_{l,e,p}$ is the capacity of an exclusive-lane lane group with protected left-turn operation (veh/h), g_l is the effective green time for the left-turn phase (s), N_l is the number of lanes in the exclusive left-turn lane group (ln), and all other variables are as previously defined.

The available capacity for the lane group is computed with Equation 31-118.

Equation 31-118

$$c_{a,l,e,p} = \frac{G_{max} \, s_{lt}}{C} N_l$$

where $c_{a,l,e,p}$ is the available capacity of an exclusive-lane lane group with protected left-turn operation (veh/h), G_{max} is the maximum green setting (s), and all other variables are as previously defined.

Equation 31-117 and Equation 31-118 can also be used to calculate the capacity of lane groups composed of through lanes and lane groups composed of right-turn lanes with proper substitution of saturation flow rate, number of lanes, and maximum green variables.

Permitted Left-Turn Operation in Exclusive Lane

The capacity for a permitted left-turn operation in an exclusive-lane lane group is computed with Equation 31-119.

Equation 31-119

$$c_{l,e} = \frac{g_u \, s_l + 3{,}600 \, n_s \, f_{ms} \, f_{sp}}{C} N_l$$

where $c_{l,e}$ is the capacity of an exclusive-lane lane group with permitted left-turn operation (veh/h), n_s is the number of sneakers per cycle = 2.0 (veh), and all other variables are as previously defined.

The available capacity for the lane group is computed with Equation 31-120.

$$c_{a,l,e} = c_{l,e} + \frac{(G_{max} - g)s_l}{C} N_l$$

Equation 31-120

where $c_{a,l,e}$ is the available capacity of an exclusive-lane lane group with permitted left-turn operation (veh/h), and all other variables are as previously defined.

The saturation flow rate s_l is specifically included in the term with the maximum green setting G_{max} in Equation 31-120 because this rate represents the saturation flow rate present at the end of the green interval. That is, it is the saturation flow rate that would occur when the green is extended to its maximum green limit as a result of cycle-by-cycle fluctuations in the demand flow rate.

Permitted Left-Turn Operation in Shared Lane

The capacity for a permitted left-turn operation in a shared-lane lane group is computed with Equation 31-121.

$$c_{sl} = \frac{g_p\, s_{sl} + 3{,}600(1 + P_L)\, f_{ms}\, f_{sp}}{C}$$

Equation 31-121

where c_{sl} is the capacity of a shared-lane lane group with permitted left-turn operation (veh/h), s_{sl} is the saturation flow rate in a shared left-turn and through lane group with permitted operation (veh/h/ln), and all other variables are as previously defined.

The saturation flow rate in Equation 31-121 is computed with Equation 31-122 (all variables are as previously defined).

$$s_{sl} = \frac{s_{th}}{g_p}\left(g_f + \frac{g_{diff}}{1 + P_L\left[\frac{E_{L2}}{f_{Lpb}} - 1\right]} + \frac{\min\,(g_p - g_f, g_u)}{1 + P_L\left[\frac{E_{L1}}{f_{Lpb}} - 1\right]} \right)$$

Equation 31-122

The available capacity for the lane group is computed with Equation 31-123.

$$c_{a,sl} = c_{sl} + \frac{(G_{max} - g_p)\, s_{sl3}}{C}$$

Equation 31-123

where $c_{a,sl}$ is the available capacity of a shared-lane lane group with permitted left-turn operation (veh/h).

The saturation flow rate s_{sl3} is specifically included in the term with the maximum green setting G_{max} in Equation 31-123 because this rate represents the saturation flow rate present at the end of the green interval.

Protected-Permitted Left-Turn Operation in Exclusive Lane

The capacity for a protected-permitted left-turn operation in an exclusive-lane lane group is computed with Equation 31-124.

$$c_{l,e,pp} = \left(\frac{g_l\, s_{lt}}{C} + \frac{g_u\, s_l + 3{,}600\, n_s\, f_{ms}\, f_{sp}}{C} \right) N_l$$

Equation 31-124

where $c_{l,e,pp}$ is the capacity of an exclusive-lane lane group with protected-permitted left-turn operation (veh/h).

The available capacity for the lane group is computed with Equation 31-125.

Equation 31-125

$$c_{a,l,e,pp} = \left(\frac{G_{max}\, s_{lt}}{C} + \frac{g_u\, s_l + 3{,}600\, n_s\, f_{ms}\, f_{sp}}{C}\right) N_l$$

where $c_{a,l,e,pp}$ is the available capacity of an exclusive-lane lane group with protected-permitted left-turn operation (veh/h) and all other variables are as previously defined.

Protected-Permitted Left-Turn Operation in Shared Lane

The capacity for a protected-permitted left-turn operation in a shared-lane lane group is computed with Equation 31-126.

Equation 31-126

$$c_{sl,pp} = \frac{g_l\, s_{sl4}}{C} + \frac{g_p\, s_{sl} + 3{,}600(1 + P_L)\, f_{ms}\, f_{sp}}{C}$$

where $c_{sl,pp}$ is the capacity of a shared-lane lane group with protected-permitted left-turn operation (veh/h).

If the lane group is associated with a leading left-turn phase, then the available capacity for the lane group is computed with Equation 31-127.

Equation 31-127

$$c_{a,sl,pp} = c_{sl,pp} + \frac{(G_{max} - g_p)\, s_{sl3}}{C}$$

where $c_{a,sl,pp}$ is the available capacity of a shared-lane lane group with protected-permitted left-turn operation (veh/h).

When the lane group is associated with a lagging left-turn phase, then the variable s_{sl3} in Equation 31-127 is replaced by s_{sl4}.

Protected-Permitted Right-Turn Operation in Exclusive Lane

The capacity for a protected-permitted right-turn operation in an exclusive-lane lane group is computed with Equation 31-128.

Equation 31-128

$$c_{r,e,pp} = \left(\frac{g_l\, s_{rt}}{C} + \frac{g_r\, s_r}{C}\right) N_r$$

where $c_{r,e,pp}$ is the capacity of an exclusive-lane lane group with protected-permitted right-turn operation (veh/h), g_l is the effective green time for the complementary left-turn phase (s), g_r is the effective green time for the phase serving the subject right-turn movement during its permitted period, and all other variables are as previously defined.

The available capacity for the lane group is computed with Equation 31-129.

Equation 31-129

$$c_{a,r,e,pp} = \left(\frac{G_{max,r}\, s_{rt}}{C} + \frac{g_r\, s_r}{C}\right) N_r$$

where $c_{a,r,e,pp}$ is the available capacity of an exclusive-lane lane group with protected-permitted right-turn operation (veh/h), and $G_{max,r}$ is the maximum green setting for the phase serving the subject right-turn movement during its permitted period (s).

4. QUEUE STORAGE RATIO

This section discusses queue storage ratio as a performance measure at a signalized intersection. This measure represents the ratio of the back-of-queue size to the available vehicle storage length. The first subsection reviews concepts related to back-of-queue estimation. The second subsection describes a procedure for estimating the back-of-queue size and queue storage ratio.

The discussion in this section describes basic principles for quantifying the back of queue for selected types of lane assignment, lane grouping, left-turn operation, and phase sequence. The analyst is referred to the computational engine for specific calculation details, especially as they relate to assignments, groupings, left-turn operation, and phase sequences not addressed in this section. This engine is described in Section 7.

CONCEPTS

The *back of queue* represents the maximum backward extent of queued vehicles during a typical cycle, as measured from the stop line to the last queued vehicle. The back-of-queue size is typically reached after the onset of the green indication. The point when it is reached occurs just before the most distant queued vehicle begins forward motion as a consequence of the green indication and in response to the forward motion of the vehicle ahead.

A *queued vehicle* is defined as a vehicle that is fully stopped as a consequence of the signal. A *full stop* is defined to occur when a vehicle slows to zero (or a crawl speed, if in queue) as a consequence of the change in signal indication from green to red, but not necessarily in direct response to an observed red indication.

The back-of-queue size that is estimated by the equations described here represents an overall average for the analysis period. It is represented in units of vehicles.

Background

Queue size is defined here to include only fully stopped vehicles. Vehicles that slow as they approach the back of the queue are considered to incur a *partial stop* but are not considered to be part of the queue. The distinction between a full and a partial stop is shown in Exhibit 31-20. This exhibit illustrates the trajectory of several vehicles as they traverse an intersection approach during one signal cycle. There is no residual queue at the end of the cycle.

Each thin line in Exhibit 31-20 that slopes upward from left to right represents the trajectory of one vehicle. The average time between trajectories represents the headway between vehicles (i.e., the inverse of flow rate q). The slope of the trajectory represents the vehicle's speed. The curved portion of a trajectory indicates deceleration or acceleration. The horizontal portion of a trajectory indicates a stopped condition. The effective red r and effective green g times are shown at the top of the exhibit. The other variables shown are defined in the discussion below.

Exhibit 31-20
Time–Space Diagram of Vehicle Trajectory on an Intersection Approach

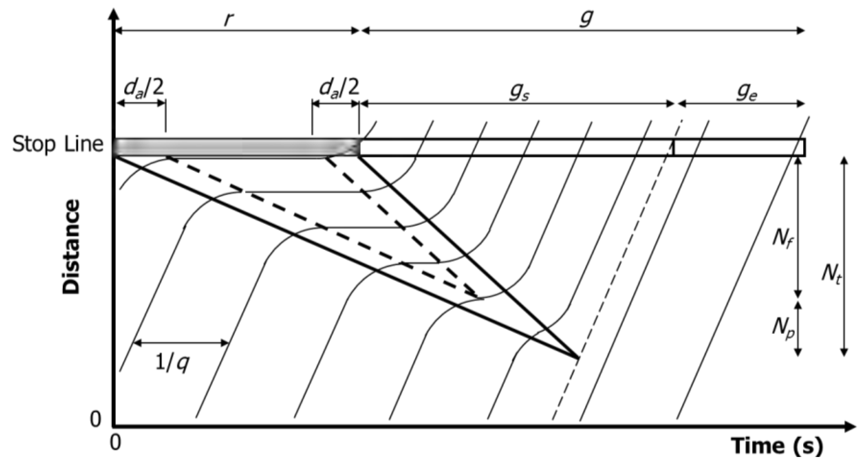


Exhibit 31-20 shows the trajectories of eight vehicles. The first five trajectories (counting from left to right) have a horizontal component to their trajectory that indicates they have reached a full stop as a result of the red indication. The sixth trajectory has some deceleration and acceleration but the vehicle does not stop. This trajectory indicates a partial stop was incurred for the associated vehicle. The last two trajectories do not incur deceleration or acceleration, and the associated vehicles do not slow or stop. Thus, the number of full stops N_f is 5 and the number of partial stops N_p is 1. The total number of stops N_t is 6. The back-of-queue size is equal to the number of full stops.

The back-of-queue size (computed by the procedure described in the next subsection) represents the average back-of-queue size for the analysis period. It is based only on those vehicles that arrive during the analysis period and join the queue. It includes the vehicles that are still in queue after the analysis period ends. The back-of-queue size for a given lane group is computed with Equation 31-130.

Equation 31-130

$$Q = Q_1 + Q_2 + Q_3$$

where

Q = back-of-queue size (veh/ln),

Q_1 = first-term back-of-queue size (veh/ln),

Q_2 = second-term back-of-queue size (veh/ln), and

Q_3 = third-term back-of-queue size (veh/ln).

The first-term back-of-queue estimate quantifies the queue size described in Exhibit 31-20. It represents the queue caused by the signal cycling through its phase sequence.

The second-term back-of-queue estimate consists of two queue components. One component accounts for the effect of random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. This fluctuation results in the occasional overflow queue at the end of the green interval (i.e., cycle failure). The second component accounts for queuing due to a sustained oversaturation during the analysis period. This queuing occurs when aggregate demand during the analysis period exceeds aggregate capacity. It is sometimes referred to as the deterministic queue component and is shown as variable $Q_{2,d}$ in Exhibit 31-21.

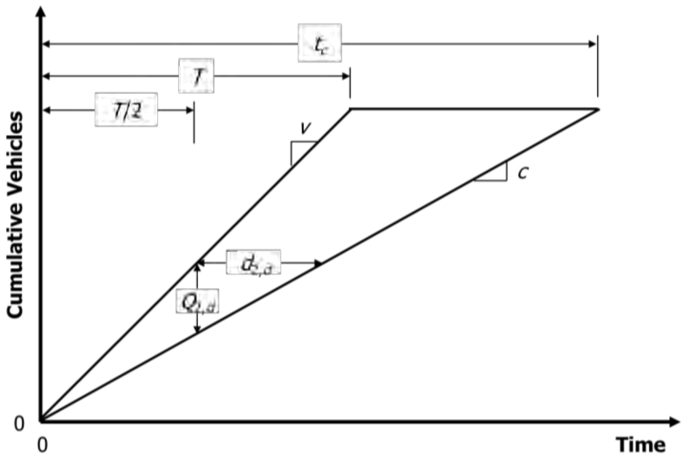


Exhibit 31-21
Cumulative Arrivals and Departures During an Oversaturated Analysis Period

Exhibit 31-21 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate v during the 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 average queue size associated with this delay is 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.*

The equation used to estimate the second-term queue is based on the assumption that no initial queue is present at the start of the analysis period. The third-term back-of-queue estimate is used to account for the additional queuing that occurs during the analysis period because of 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 31-22 illustrates the queue due to an initial queue as a trapezoid shape bounded by thick lines. The average queue is represented by the variable Q_3. The initial queue size is shown as consisting of 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 31-22. However, it can be less than the analysis period duration for some lower-volume conditions.

Exhibit 31-22
Third-Term Back-of-Queue Size with Increasing Queue

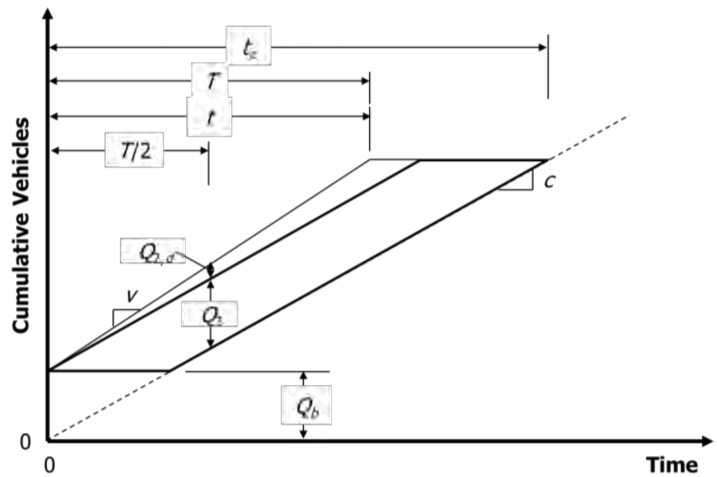


Exhibit 31-22 illustrates the case in which the demand flow rate v exceeds the capacity c during the analysis period. In contrast, Exhibit 31-23 and Exhibit 31-24 illustrate alternative cases in which the demand flow rate is less than the capacity.

Exhibit 31-23
Third-Term Back-of-Queue Size with Decreasing Queue

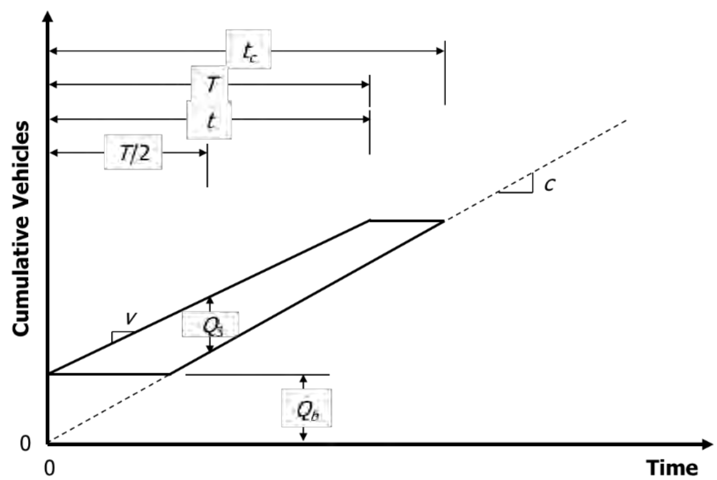
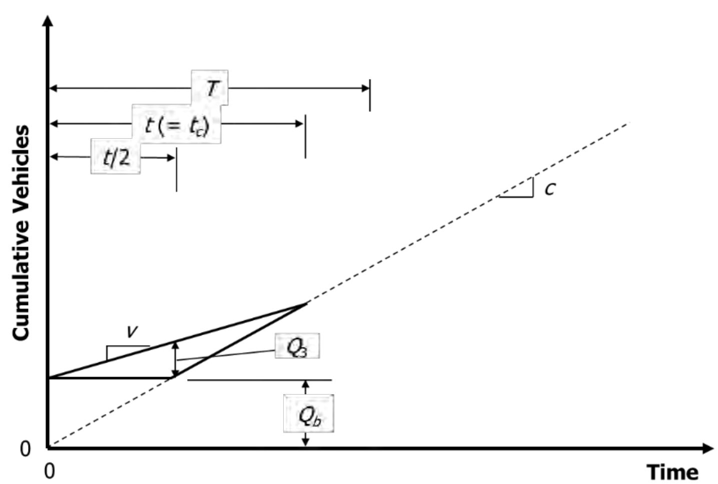


Exhibit 31-24
Third-Term Back-of-Queue Size with Queue Clearing



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

Acceleration–Deceleration Delay

The acceleration–deceleration delay d_a term shown in Exhibit 31-20 is used to distinguish between a fully and a partially stopped vehicle. This delay term represents the time required to decelerate to a stop and then accelerate back to the initial speed, less the time it would have taken to traverse the equivalent distance at the initial speed.

Various definitions are used to describe when a vehicle is stopped for the purpose of field measurement. These definitions typically allow the observed vehicle to be called "stopped" even if it has a slow speed (e.g., 2 to 5 mi/h) while moving up in the queue. Many stochastic simulation programs also have a similar allowance. These practical considerations in the count of stopped vehicles require the specification of a threshold speed that can be used to identify when a vehicle is effectively stopped. The acceleration–deceleration delay for a specified threshold speed is estimated with Equation 31-131.

$$d_a = \frac{[1.47\ (S_a - S_s)]^2}{2\ (1.47\ S_a)} \left(\frac{1}{r_a} + \frac{1}{r_d}\right)$$

Equation 31-131

where

d_a = acceleration–deceleration delay (s),

S_a = average speed on the intersection approach (mi/h),

S_s = threshold speed defining a stopped vehicle = 5.0 (mi/h),

r_a = acceleration rate = 3.5 (ft/s^2), and

r_d = deceleration rate = 4.0 (ft/s^2).

The average speed on the intersection approach S_a is representative of vehicles that would pass unimpeded through the intersection if the signal were green for an extended period. It can be estimated with Equation 31-132.

$$S_a = 0.90\ (25.6 + 0.47\ S_{pl})$$

Equation 31-132

where S_{pl} is the posted speed limit (mi/h).

The threshold speed S_s represents the speed at or below which a vehicle is said to be effectively stopped while in queue or when joining a queue. The strictest definition of this speed is 0.0 mi/h, which coincides with a complete stop. However, vehicles sometimes move up in the queue while drivers wait for the green indication. A vehicle that moves up in the queue and then stops again does not incur an additional full stop. The threshold speed that is judged to differentiate between vehicles that truly stop and those that are just moving up in the queue is 5 mi/h.

Acceleration–deceleration delay values from Equation 31-131 typically range from 8 to 14 s, with larger values in this range corresponding to higher speeds.

Arrival–Departure Polygon

The arrival–departure polygon (ADP) associated with a lane is a graphic tool for computing the number of full stops N_f. The number of full stops has been shown to be equivalent to the first-term back-of-queue size (*5*).

The ADP separately portrays the cumulative number of arrivals and departures associated with a traffic movement as a function of time during the average cycle. It is related but not identical to the QAP. The main difference is that the polygon sides in the ADP represent an arrival rate or a discharge rate but not both. In contrast, the polygon sides in the QAP represent the combined arrival and discharge rates that may occur during a common time interval.

The ADP is useful for estimating the stop rate and back-of-queue size, and the QAP is useful for estimating delay and queue service time.

The ADP for a through movement is presented in Exhibit 31-25, which shows the polygon for a typical cycle. The red and green intervals are ordered from left to right in the sequence of presentation so that the last two time periods correspond to the queue service time g_s and green extension time g_e of the subject phase. The variables shown in the exhibit are defined in the following list:

t_f = service time for fully stopped vehicles (s),

N_f = number of fully stopped vehicles (veh/ln),

g_s = queue service time (s),

g_e = green extension time (s),

q_r = arrival flow rate during the effective red time = $(1 - P)\, q\, C/r$ (veh/s),

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

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

v = demand flow rate (veh/h),

r = effective red time = $C - g$ (s),

g = effective green time (s),

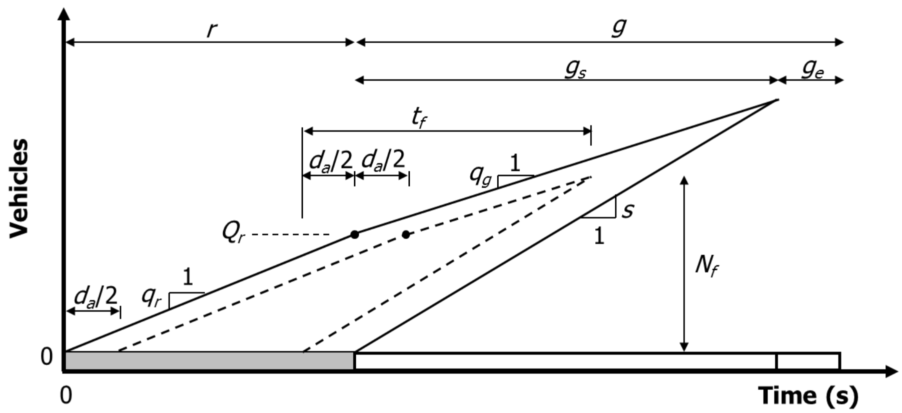
C = cycle length (s),

q_g = arrival flow rate during the effective green time = $P\, q\, C/g$ (veh/s), and

Q_r = queue size at the end of the effective red time = $q_r\, r$ (veh).

In application, all flow rate variables are converted to common units of vehicles per second per lane. The presentation in this section is based on these units for q and s. If the flow rate q exceeds the lane capacity, then it is set to equal this capacity.

Exhibit 31-25
Arrival–Departure Polygon



The upper solid trend line in Exhibit 31-25 corresponds to vehicles arriving at the intersection. The lower solid trend line corresponds to queued vehicles departing the stop line. The lower trend line is horizontal during the effective red, denoting no departures. The vertical distance between these two lines at any instant in time represents the number of vehicles in the queue.

At the start of the effective red, vehicles begin to queue at a rate of q_r and accumulate to a length of Q_r vehicles at the time the effective green begins. Thereafter, the rate of arrival is q_g until the end of the effective green period. The queue service time g_s represents the time required to serve the queue present at the end of the effective red Q_r plus any additional arrivals that join the queue before it fully clears. The dashed line in this exhibit represents only those vehicles that complete a full stop. The dashed line lags behind the solid arrival line by one-half the value of d_a (i.e., $d_a/2$). In contrast, the dashed line corresponding to initiation of the departure process leads the solid departure line by $d_a/2$.

One-half the acceleration–deceleration delay d_a (i.e., $d_a/2$) occurs at both the end of the arrival process and the start of the discharge process. This assumption is made for convenience in developing the polygon. The derivation of the stop rate and queue length equations indicates that the two components are always combined as d_a. Thus, the assumed distribution of this delay to each of the two occurrences does not influence the accuracy of the estimated back-of-queue size.

The number of fully stopped vehicles N_f represents the number of vehicles that arrive before the queue of stopped vehicles has departed. Equation 31-133 is used for computing this variable (all other variables are as previously defined).

Equation 31-133

$$N_f = q_r\, r + q_g\left(t_f - d_a\right)$$

Equation 31-134 can also be used for estimating N_f.

Equation 31-134

$$N_f = \frac{s\, t_f}{3{,}600}$$

Combining Equation 31-133 and Equation 31-134 to eliminate N_f and solve for t_f yields Equation 31-135.

Equation 31-135

$$t_f = \frac{q_r\, r - q_g\, d_a}{s - q_g}$$

Equation 31-135 can be used with Equation 31-133 to obtain an estimate of N_f. The first-term back-of-queue size is then computed with Equation 31-136.

Equation 31-136

$$Q_1 = N_f$$

The polygon in Exhibit 31-25 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. Other shapes are possible, depending on whether the lane group includes a shared lane and whether the lane group serves a permitted (or protected-permitted) left-turn movement. In general, a unique shape is dictated by each combination of left-turn operational mode (i.e., permitted, protected, or protected-permitted) and phase sequence (i.e., lead, lag, or split). A general procedure for constructing these polygons is described in the next subsection.

PROCEDURE FOR ESTIMATING BACK OF QUEUE FOR SELECTED LANE GROUPS

This subsection describes a procedure for estimating the back-of-queue size for a lane group at a signalized intersection. The procedure is described in a narrative format and does not define every equation needed to develop a polygon for every combination of lane allocation, left-turn operational mode, and phase sequence. This approach is taken because of the large number of equations required to address the full range of combinations found at intersections in most cities. However, all these equations have been developed and are automated in the computational engine that is described in Section 7. Some of the equations presented in the previous section are repeated in this subsection for reader convenience.

The procedure requires the previous construction of the QAP. The construction of the QAP is described in Section 3.

Step 1. Determine Acceleration–Deceleration Delay

The acceleration–deceleration delay term is used to distinguish between fully and partially stopped vehicles. It is computed with Equation 31-131.

Step 2. Define Arrival–Departure Polygon

During this step, the green times and flow rates used previously to construct the QAP are now used to construct the ADP associated with each lane group served during a phase.

The ADP in Exhibit 31-25 applies to either a through lane group or a left- or right-turn lane group with exclusive lanes operating with the protected mode. This polygon is also applicable to split phasing and to shared lane groups serving through and right-turning vehicles operating with the permitted mode. For split phasing, each approach is evaluated separately to determine its overall stop rate. If the approach has a turn lane, then a separate polygon is constructed for both the turn and the through lane groups.

More complicated combinations of phase sequence and left-turn operational mode dictate more complicated polygons. A polygon must be derived for each combination. The most common combinations are illustrated in Exhibit 31-26 through Exhibit 31-29.

The concept is extended to shared left-turn and through lane groups with protected-permitted operation in Exhibit 31-30 and Exhibit 31-31. Other polygon shapes exist, depending on traffic flow rates, phase sequence, lane use, and left-turn operational mode. The concept of construction must be extended to these other shapes to estimate accurately the back-of-queue size.

Most variables shown in these exhibits were defined in previous subsections. The following variables are also defined:

g_p = effective green time for permitted left-turn operation (s),

g_u = duration of permitted left-turn green time that is not blocked by an opposing queue (s),

g_f = time before the first left-turning vehicle arrives and blocks the shared lane (s),

g_l = effective green time for left-turn phase (s),

g_{ps} = queue service time during permitted left-turn operation (s),

s_p = saturation flow rate of a permitted left-turn movement (veh/h/ln),

s_{lt} = saturation flow rate of an exclusive left-turn lane with protected operation = s_{th}/E_L (veh/h/ln),

E_L = equivalent number of through cars for a protected left-turning vehicle = 1.05,

s_{th} = saturation flow rate of an exclusive through lane (= base saturation flow rate adjusted for lane width, heavy vehicles, grade, parking, buses, and area type) (veh/h/ln), and

P_L = proportion of left-turning vehicles in the shared lane (decimal).

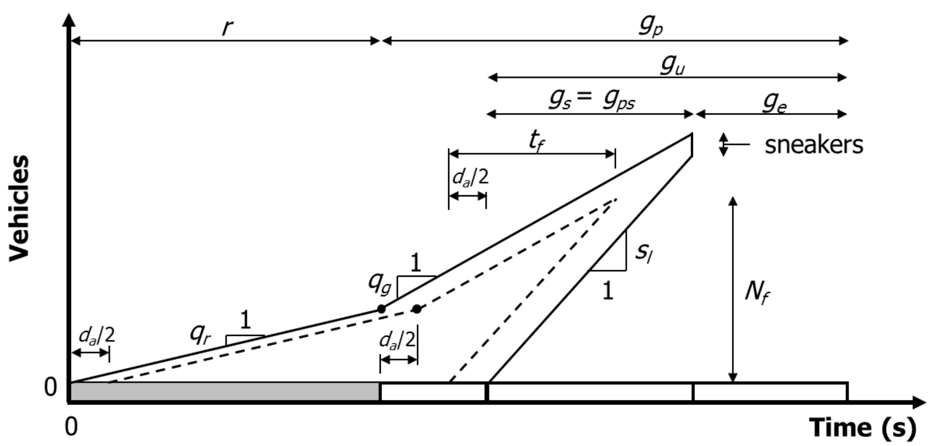


Exhibit 31-26
ADP for Permitted Left-Turn Operation in an Exclusive Lane

Exhibit 31-27
ADP for Permitted Left-Turn Operation in a Shared Lane

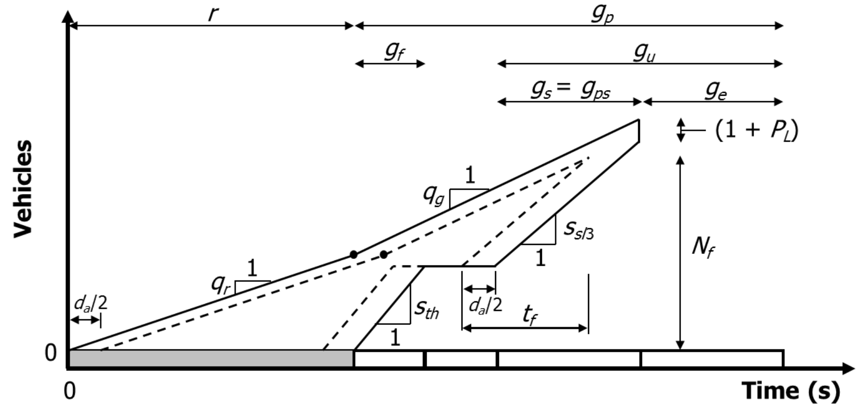


Exhibit 31-28
ADP for Leading, Protected-Permitted Left-Turn Operation in an Exclusive Lane

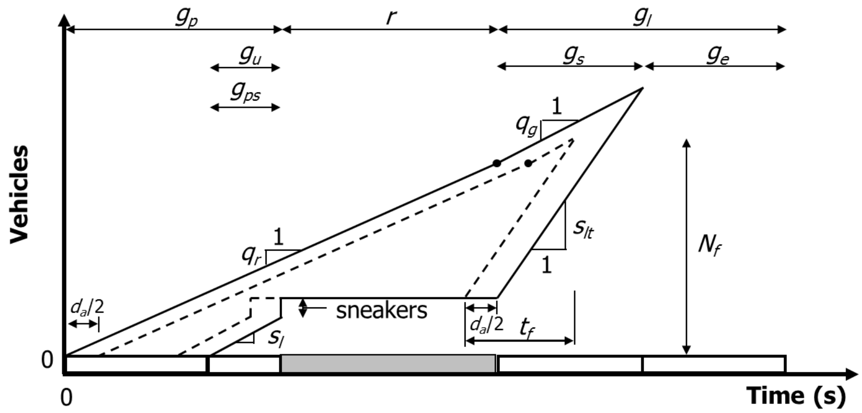
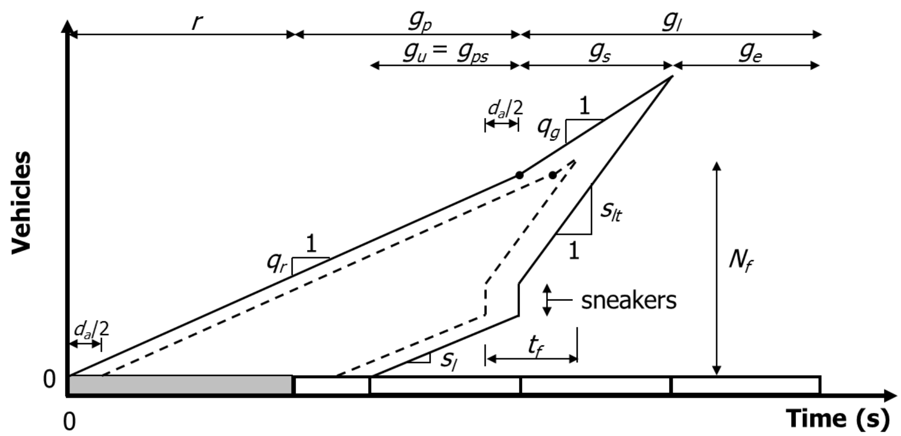


Exhibit 31-29
ADP for Lagging, Protected-Permitted Left-Turn Operation in an Exclusive Lane



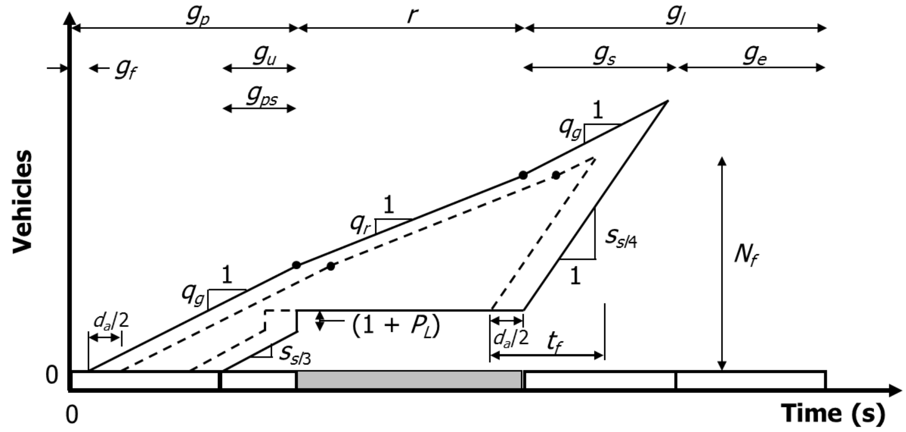


Exhibit 31-30
ADP for Leading, Protected-Permitted Left-Turn Operation in a Shared Lane

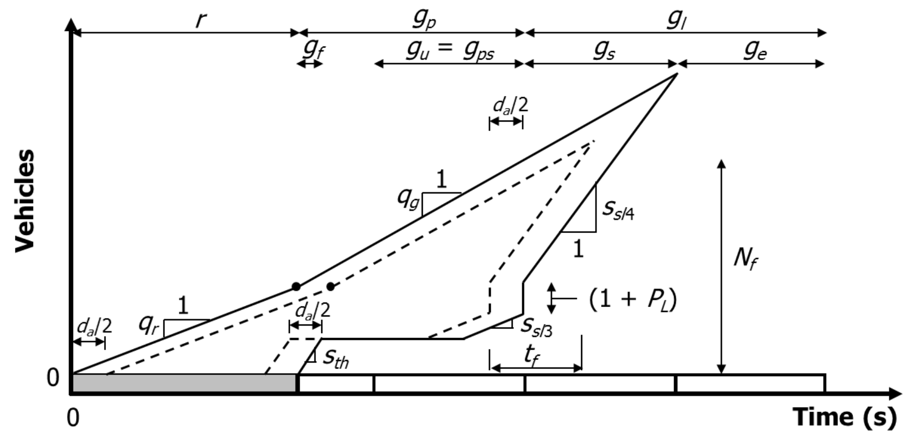


Exhibit 31-31
ADP for Lagging, Protected-Permitted Left-Turn Operation in a Shared Lane

The polygon in Exhibit 31-26 applies to the left-turn lane group served by an exclusive lane that operates in the permitted mode during the adjacent through phase. If the phase extends to max-out, then some left-turning vehicles will be served as sneakers. The expected number of sneakers for this mode is reduced if downstream lane blockage or spillback is present [i.e., sneakers = $n_s f_{ms} f_{sp}$, where n_s is the number of sneakers per cycle = 2.0 (veh), f_{ms} is the adjustment factor for downstream lane blockage, and f_{sp} is the adjustment factor for sustained spillback].

The polygon in Exhibit 31-27 applies to the left-turn and through lane group on a shared-lane approach with permitted operation. If the phase extends to max-out, then some left-turning vehicles will be served as sneakers. The expected number of sneakers (shown as $1 + P_L$) is computed as $(1 + P_L) f_{ms} f_{sp}$, where P_L is the proportion of left-turning vehicles in the shared lane, and all other variables are as previously defined.

The polygon in Exhibit 31-28 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and an exclusive left-turn lane. The polygon in Exhibit 31-29 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the number of sneakers (where sneakers = $n_s f_{ms} f_{sp}$).

The polygon in Exhibit 31-30 applies to left-turn movements that have protected-permitted operation with a leading left-turn phase and a shared left-turn and through lane group. The polygon in Exhibit 31-31 applies to the same movements and operation but with a lagging left-turn phase. If a queue exists at the end of the permitted period for either polygon, then the queue is reduced by the expected number of sneakers [which is computed as $(1 + P_L) f_{ms} f_{sp}$].

As noted above, all polygons are based on the requirement that lane volume cannot exceed lane capacity for the purpose of estimating the queue service time. This requirement is met in the polygons shown because the queue size equals 0.0 vehicles at some point during the cycle.

Step 3. Define Arrival–Departure Polygon for Fully Stopped Vehicles

During this step, the polygon defined in the previous step is enhanced to include the polygon shape for the fully stopped vehicles. The fully stopped vehicle polygon is defined by dashed lines in Exhibit 31-25 through Exhibit 31-31.

Two rules guide the development of this polygon feature. First, the dashed line that corresponds to arrivals at the stopped queue *lags* behind the solid arrival line by $d_a/2$ s. Second, the dashed line that corresponds to initiation of the departure process *leads* the solid departure line by $d_a/2$ s.

Step 4. Compute Service Time for Fully Stopped Vehicles

The service time t_f is computed for each polygon constructed in the previous step. When the polygon in Exhibit 31-25 applies, then either Equation 31-137 or Equation 31-138 can be used to compute this time.

If $d_a \leq (1 - P)\, g\, X$, then

Equation 31-137

$$t_f = \frac{q\,C\,(1 - P - P\,d_a/g)}{s\,[1 - \min(1, X)\,P]}$$

otherwise

Equation 31-138

$$t_f = \frac{q\,C\,(1 - P)(r - d_a)}{s\,[r - \min(1, X)\,(1 - P)g]}$$

where X is the volume-to-capacity ratio.

The saturation flow rate s used in Equation 31-137 and Equation 31-138 represents the adjusted saturation flow rate that is computed by the procedure described in Section 3 of Chapter 19, Signalized Intersections.

Step 5. Compute the Number of Fully Stopped Vehicles

The number of fully stopped vehicles N_f is computed for each polygon constructed in Step 3. When the polygon in Exhibit 31-25 applies, then Equation 31-139 or Equation 31-140 can be used to compute the number of stops.

If $d_a \leq (1 - P)\, g\, X$, then

Equation 31-139

$$N_f = q_r\, r + q_g(t_f - d_a)$$

otherwise

Equation 31-140

$$N_f = q_r\,(r - d_a + t_f)$$

Step 6. Compute the First-Term Back-of-Queue Size

The first-term back-of-queue estimate Q_1 (in vehicles per lane) is computed by using the number of fully stopped vehicles from the previous step. It is computed with Equation 31-141, where N_f is the number of fully stopped vehicles.

$$Q_1 = N_f$$ Equation 31-141

For some of the more complex ADPs that include left-turn movements operating with the permitted mode, the queue may dissipate at two or more points during the cycle. If this occurs, then $N_{f,i}$ is computed for each of the i periods between queue dissipation points. The first-term back-of-queue estimate is then equal to the largest of the $N_{f,i}$ values computed in this manner.

Step 7. Compute the Second-Term Back-of-Queue Size

Equation 31-142 is used to compute the second-term back-of-queue estimate Q_2 for lane groups served by an actuated phase.

$$Q_2 = \frac{c_A}{3{,}600\ N} d_2$$ Equation 31-142

where

Q_2 = second-term back-of-queue size (veh/ln),

c_A = average capacity (veh/h),

d_2 = incremental delay (s/veh), and

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

If there is no initial queue, then the average capacity c_A is equal to the lane group capacity c. The procedure for computing this capacity is described in Section 3 of Chapter 19. If there is an initial queue, then the average capacity is computed with the procedure described in Section 4 of Chapter 19.

Step 8. Compute the Third-Term Back-of-Queue Size

The third-term back-of-queue estimate Q_3 is calculated with Equation 31-143 through Equation 31-148.

$$Q_3 = \frac{1}{N\ T}\left(t_A \frac{Q_b + Q_e - Q_{eo}}{2}\right)$$ Equation 31-143

with

$$Q_e = Q_b + t_A(v - c_A)$$ Equation 31-144

If $v \geq c_A$, then

$$Q_{eo} = T(v - c_A)$$ Equation 31-145

$$t_A = T$$ Equation 31-146

If $v < c_A$, then

$$Q_{eo} = 0.0 \text{ veh}$$ Equation 31-147

$$t_A = Q_b/(c_A - v) \leq T$$ Equation 31-148

where

Q_3 = third-term back-of-queue size (veh/ln),

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

T = analysis period duration (h),

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

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

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

Step 9. Compute the Back-of-Queue Size

The average back-of-queue estimate Q for a lane group (in vehicles per lane) is computed with Equation 31-149 (all other variables are as previously defined).

Equation 31-149

$$Q = Q_1 + Q_2 + Q_3$$

If desired, a percentile back-of-queue estimate $Q_{\%}$ can be computed with Equation 31-150, and Equation 31-151 through Equation 31-153 can be used to compute the percentile back-of-queue factor $f_{B\%}$.

Equation 31-150

$$Q_{\%} = (Q_1 + Q_2) f_{B\%} + Q_3$$

with

If $v \geq c_A$, then

Equation 31-151

$$f_{B\%} = \min\left(1.8, 1.0 + z\sqrt{\frac{I}{Q_1 + Q_2}} + 0.60\, z^{0.24} \left(\frac{g}{C}\right)^{0.33} (1.0 - e^{2-2\,X_A})\right)$$

Equation 31-152

$$X_A = v/c_A$$

If $v < c_A$, then

Equation 31-153

$$f_{B\%} = \min\left(1.8, 1.0 + z\sqrt{\frac{I}{Q_1 + Q_2}}\right)$$

where

$Q_{\%}$ = percentile back-of-queue size (veh/ln);

$f_{B\%}$ = percentile back-of-queue factor;

z = percentile parameter = 1.04 for 85th percentile queue, 1.28 for 90th percentile queue, and 1.64 for 95th percentile queue;

I = upstream filtering adjustment factor; and

X_A = average volume-to-capacity ratio.

Step 10. Compute Queue Storage Ratio

If the lane group is served by a bay or lane of limited storage length, then the queue storage ratio can be computed by using Equation 31-154 with Equation 31-155.

$$R_Q = \frac{L_h\, Q}{L_a}$$ Equation 31-154

with

$$L_h = L_{pc}(1 - 0.01\, P_{HV}) + 0.01\, L_{HV} P_{HV}$$ Equation 31-155

where

R_Q = queue storage ratio,

L_a = available queue storage distance (ft/ln),

L_h = average vehicle spacing in stationary queue (ft/veh),

L_{pc} = stored passenger car lane length = 25 (ft),

L_{HV} = stored heavy-vehicle lane length = 45 (ft), and

P_{HV} = percentage heavy vehicles in the corresponding movement group (%).

Average vehicle spacing is the average length between the front bumpers of two successive vehicles in a stationary queue. The available queue storage distance is equal to the turn bay (or lane) length.

The queue storage ratio is useful for quantifying the potential blockage of the available queue storage distance. If the queue storage ratio is less than 1.0, then blockage will not occur during the analysis period. Blockage will occur if the queue storage ratio is equal to or greater than 1.0.

If desired, a percentile queue storage ratio can be computed with Equation 31-156.

$$R_{Q\%} = \frac{L_h\, Q_{\%}}{L_a}$$ Equation 31-156

where $R_{Q\%}$ is the percentile queue storage ratio.

5. PLANNING-LEVEL ANALYSIS APPLICATION

The planning-level analysis application described in this section is intended to provide the user a means for conducting a simplified and approximate analysis of signalized intersection operations for motorized vehicles. Chapter 19, Signalized Intersections, provides a more detailed methodology. The objective of the planning-level analysis application is to assess whether an intersection's geometric conditions are sufficient to handle the projected demand volume. Within this framework, many of the data required for a full operational analysis are not needed. This method has several potential uses and applications:

- Conducting sketch-level analyses to quickly assess whether an intersection's lane geometry is sufficient to accommodate a given set of turn-movement demand volumes;
- Evaluating intersection geometry and lane widening alternatives;
- Estimating signal phasing and timing;
- Comparing analysis results against traffic operational performance results produced by other methods; and
- Educating students, transportation professionals, and nontransportation professionals about the fundamentals of traffic signal operational performance.

OVERVIEW OF THE APPLICATION

This subsection provides an overview of the two parts of the planning-level analysis application. Part I provides an estimate of intersection capacity sufficiency. Part II extends the analysis from Part I to provide an estimate of delay and level of service (LOS).

The planning-level analysis application is designed to evaluate the performance of designated groups of lanes, an intersection approach, and the entire intersection. A group of lanes designated for separate analysis is referred to as a *lane group*. Lane groups form the basis for intersection analysis in the planning-level analysis application and in the motorized vehicle methodology described in Chapter 19. However, the criteria for defining a lane group are different between the two methodologies.

For the planning-level analysis application, all traffic movements for a given approach (i.e., left, through, and right) must be assigned to at least one lane group. A lane group can consist of one or more lanes. There are two guidelines to follow for assigning traffic movements to lane groups:

1. When a traffic movement uses only an exclusive lane (or lanes), it is analyzed as an exclusive lane group.
2. When two or more traffic movements share a lane, all lanes that convey those traffic movements are analyzed as a mixed lane group.

When a right-turn movement is shared with a through movement, it is considered to be a part of the through movement lane group. When a right-turn movement is shared with a left-turn movement (such as at a T-intersection), it is considered to be a part of the left-turn movement lane group. The concept of lane group is discussed in more detail in the Methodology subsection.

Part I: Intersection Sufficiency Assessment

Part I provides an estimate of the intersection's volume-to-capacity ratio, which can be used to assess whether the intersection is likely to operate under, near, or over capacity during the analysis period. This assessment is predicated on the critical movement analysis technique developed originally as part of *Transportation Research Circular 212* (*6*).

Part I generally requires only two inputs: turn movement volume and lane geometry. Other input data are allowed, but they can also be set to default values if they are not explicitly known. Part I can be applied by using manual calculations; it does not require software to implement.

Part I consists of the following steps:

1. Determine left-turn operation.
2. Convert movement volumes to through passenger-car equivalents.
3. Assign flow rates to lane groups.
4. Determine critical lane groups.
5. Determine intersection sufficiency.

Part II: Delay and Level of Service Assessment

Part II extends the results from Part I to produce estimates of volume-to-capacity ratio, delay, and LOS. For practical purposes, Part II requires a spreadsheet or other software to compute estimates of delay and LOS. A Part II analysis requires the initial completion of Steps 1 to 5 of Part I. It then continues with the following steps:

6. Calculate capacity.
7. Determine delay and LOS.

Limitations

The planning-level analysis application has the following limitations:

- It only considers the performance of motorized vehicles;
- It is based on pretimed operation and thus does not account for the effects of actuated control;
- It does not analyze all potential combinations of left-turn operation for opposing approaches (e.g., protected left-turn operation opposed by permitted left-turn operation is not addressed by the application);
- It does not explicitly consider the effects of poorly timed signals;
- It does not account for upstream or downstream impedances and effects of short lanes; and
- It does not consider the effects of grade, lane width, bus activity, area type, pedestrian–vehicle conflicts, or pedestrian–bicycle conflicts;

however, an "equivalency factor for other conditions" is provided to allow the analyst to account for these (or other) nonideal conditions.

REQUIRED DATA AND SOURCES

Exhibit 31-32 describes the input data requirements for conducting an analysis using the planning-level analysis application.

Exhibit 31-32
Required Input Data for the Planning-Level Analysis Application

Data Item	Comments
Part I	
Number of lanes and lane use	Required. Exclusive or shared lane use.
Turn movement volumes	Required
Intersection peak hour factor	Use default value of 0.92 if not known.
Percentage heavy vehicles	Use default value of 3% if not known.
On-street parking presence	No (default)
Level of pedestrian activity	None (default) Low – 50 p/h Medium – 200 p/h High – 400 p/h Extreme – 800 p/h
Left-turn operation and phase sequence	Protected operation—with left-turn phase Permitted operation—no left-turn phase Protected operation—split phasing Protected-permitted operation–with left-turn phase (Can be estimated—use guidance provided in the application)
Base saturation flow rate	(Can be estimated—use guidance provided in the application)
Cycle length	(Can be estimated—use guidance provided in the application)
Effective green time	Required to evaluate protected-permitted operation, if present (Can be estimated—use guidance provided in the application)
Part II	
Effective green time	(Can be estimated—use guidance provided in the application)
Progression quality	Good progression Random arrivals (default) Poor progression

The analyst is required to specify values for two data items: (*a*) the volume for each movement and (*b*) the number of lanes (and the turn designation for each lane) on each approach. The effective green time is also required if protected-permitted left-turn operation is to be evaluated. Default values can be assumed for the other input data, or the user can specify these values if they are known.

METHODOLOGY

Part I: Intersection Sufficiency Assessment

The first part of the application consists of five steps. These steps are completed in sequence to evaluate the capacity sufficiency of the intersection.

Step 1: Determine Left-Turn Operation

For approaches with left-turn movements, the left-turn operational mode and phase sequence must be defined. The following mode and sequence combinations are addressed in the planning-level analysis application:

- **Protected operation—with left-turn phase.** This combination enables the subject left-turn movement to proceed concurrently with either the adjacent through movement or the opposing left-turn movement.

- **Permitted operation—no left-turn phase.** This combination enables the subject left-turn movement to proceed through the intersection during the same phase indication as the opposing through movement. It generally results in higher capacity for the intersection than other combinations. However, it also produces the highest potential safety conflicts.
- **Protected operation—split phasing.** With split phasing, the through and left-turn movements on the subject approach are served in a protected manner during a common phase. This combination is generally the least efficient type of operation and is oftentimes used when geometric properties of the intersection preclude movements on opposing approaches from proceeding at the same time, or when traffic volumes on opposite approaches are unbalanced.
- **Protected-permitted operation—with left-turn phase.** This combination serves left turns in a protected manner during a left-turn phase and in a permitted manner during a through phase. If this combination is to be evaluated, the analyst should refer to the supplemental procedure in the Protected-Permitted Left-Turn Operations section.

If the operational mode is not known, the following general rules can be applied to determine if protected operation is appropriate for planning-level analysis purposes. Protected operation should be assumed if any of the following conditions are met:

1. The left-turn volume is greater than or equal to 240 veh/h.
2. The product of the left-turn volume and the opposing through volume exceeds a given threshold (50,000 if there is one opposing through lane, 90,000 if there are two opposing through lanes, and 110,000 if there are three or more opposing through lanes).
3. There is more than one left-turn lane on the approach.

Several other considerations for choosing a left-turn operation are not considered to be an explicit part of a planning method. The *Traffic Engineering Handbook* (*7*) provides additional criteria that include the speed of vehicles on the opposing approach, restrictive sight distances, and accident rates, among others. Therefore, protected left-turn operation may be appropriate even when the above conditions are not satisfied.

In some cases, an intersection may have protected left-turn operation on one approach and permitted left-turn operation on the opposite approach. When this situation occurs, it is necessary to assume both approaches have protected operation to use the planning-level analysis application.

Step 2: Convert Movement Volumes to Through Passenger-Car Equivalents

The objective of this step is to convert all movement volumes into through passenger-car equivalents. The conversion considers one or more of the following factors:

- Effect of heavy vehicles,
- Variation in flow during the hour,

- Impact of opposing through vehicles on permitted left-turn vehicles,
- Impact of pedestrians on right-turn vehicles,
- Impact of parking maneuvers, and
- Lane utilization.

Equation 31-157 provides the volume adjustment equation. Each of the factors in this equation is described in the subsequent paragraphs.

Equation 31-157

$$v_{adj} = V\, E_{HV}\, E_{PHF}\, E_{LT}\, E_{RT}\, E_p\, E_{LU}\, E_{\text{other}}$$

where

v_{adj} = equivalent through movement flow rate expressed in through passenger cars per hour (tpc/h),

V = movement volume (veh/h),

E_{HV} = equivalency factor for heavy vehicles,

E_{PHF} = equivalency factor for peaking characteristics,

E_{RT} = equivalency factor for right turns,

E_{LT} = equivalency factor for left turns,

E_p = equivalency factor for parking activity,

E_{LU} = equivalency factor for lane utilization, and

E_{other} = equivalency factor for other conditions.

Adjustment for Heavy Vehicles

The equivalency factor to convert the mixed traffic stream into passenger car equivalents is computed with Equation 31-158.

Equation 31-158

$$E_{HV} = 1 + 0.01\, P_{HV}(E_T - 1)$$

where

P_{HV} = percentage of heavy vehicles in the corresponding lane group (%), and

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

The recommended passenger car equivalent E_T in this method is 2.0. If the user has more detailed or localized information about the value of E_T, then this value may be used in Equation 31-158.

Adjustment for Variation in Flow During the Hour

The movement volume is adjusted by the peak hour factor to reflect the peak 15-min flow rate, similar to the procedure used in the operational method. Equation 31-159 is used to compute the peak hour adjustment factor.

Equation 31-159

$$E_{PHF} = \frac{1}{PHF}$$

where PHF is the peak hour factor (varies between 0.25 and 1.00).

Adjustment for Impedances Experienced by Turning Vehicles

The equivalency factors used to account for impedances experienced by left- and right-turn movements are shown in Exhibit 31-33 and Exhibit 31-34.

Exhibit 31-33
Planning-Level Analysis: Equivalency Factor for Left Turns

Left-Turn Operation	Total Opposing Volume V_o (veh/h)[a]	Equivalency Factor for Left Turns E_{LT}
Protected—with left-turn phase Protected—split phasing	Any	1.05
Permitted—no left-turn phase	<200	1.1
	200–599	2.0
	600–799	3.0
	800–999	4.0
	≥1,000	5.0
Protected-permitted—with left-turn phase	Refer to guidance in the Protected-Permitted Left-Turn Operations section	

Note: [a] Includes the sum of through and right-turn volumes on the opposing approach, regardless of whether the right-turn volume is served in an exclusive right-turn lane.

Exhibit 31-34
Planning-Level Analysis: Equivalency Factor for Right Turns

Level of Pedestrian Activity	Pedestrian Volume (p/h)	Equivalency Factor for Right Turns E_{RT}
None or low	0–199	1.2
Moderate	200–399	1.3
High	400–799	1.5
Extreme	≥800	2.1

In Exhibit 31-33, the equivalency factor that is applicable to permitted left-turn movements is based on the opposing volume. This volume is defined as the sum of opposing through and right-turn movements, regardless of whether the right-turn volume is served in an exclusive right-turn lane. The equivalency factor for right turns is a function of the pedestrian activity in the crosswalk that conflicts with the subject right-turn movement.

Adjustment for Parking Activity

The equivalency factor for on-street parking activity is shown in Exhibit 31-35. This factor is applicable to through and right-turn vehicles. It is also applicable to left-turn vehicles on a one-way street when parking is allowed on the left side.

Exhibit 31-35
Planning-Level Analysis: Equivalency Factor for Parking Activity

On-Street Parking Presence	No. of Lanes in Lane Group	Equivalency Factor for Parking Activity E_p
No	All	1.00
Yes	1	1.20
	2	1.10
	3	1.05

Adjustment for Lane Utilization

The planning-level analysis application analyzes the performance of the heaviest-traveled lane. For lane groups with two or more lanes, the volume is adjusted to reflect the heaviest-traveled lane. The appropriate equivalency factor to account for lane utilization is selected from Exhibit 31-36.

Exhibit 31-36
Planning-Level Analysis: Equivalency Factor for Lane Utilization

Lane Group Movement	No. of Lanes in Lane Group	Equivalency Factor for Lane Utilization E_{LU}
Through or shared	1	1.00
	2	1.05
	≥3	1.10
Exclusive left turn	1	1.00
	≥2	1.03
Exclusive right turn	1	1.00
	≥2	1.13

Adjustment for "Other" Conditions

An adjustment factor for "other" is provided in Equation 31-157. This factor is a placeholder to allow the user to further adjust the movement volume for conditions that are not captured by any other adjustment factor. The analyst may apply any combination of the saturation flow rate adjustment factors presented in Section 3 of Chapter 19 to reflect other nonideal conditions. In this situation, E_{other} is computed as the product of the inverted factors (i.e., $E_{\text{other}} = 1/f_i \times 1/f_j \times \ldots \times 1/f_n$, where f_i, f_j, and f_n represent the factors in Chapter 19 that are applicable to the subject movement).

Step 3: Assign Flow Rates to Lane Groups

Initially, lane groups should be checked to determine if a de facto turn lane exists. A de facto turn lane occurs on approaches with multilane lane groups where (*a*) either a left- or right-turn movement is shared with a through movement and (*b*) the turning flow rates are sufficiently high, or the impedance to the turning traffic is sufficiently great, to reasonably expect that the through vehicles use only the adjacent exclusive through lane(s) and avoid the shared lane.

The presence of a de facto turn lane can be determined by comparing the total flow rate of turning traffic (left or right) with the lane-equivalent adjusted flow rate in the shared lane as calculated in Step 2. If the flow rate of turning traffic is greater than the lane-equivalent adjusted flow rate, a de facto turn lane should be assumed. De facto turn lanes should be analyzed as exclusive turn lanes, and thus all through movements should be assigned to the through-only lane(s).

In cases in which there are multiple turn lanes and one lane is shared with a through movement, these lanes should be treated as a single lane group that is designated as the through lane group. For approaches at a T-intersection where there are only left- and right-turn movements and multiple lanes, and one of the lanes is shared, the user has the option of coding all lanes as either the right-turn lane group or the left-turn lane group.

Once lane groups have been defined, the lane group flow rate is divided by the number of lanes associated with the lane group to obtain the lane flow rate. Equation 31-160 is used for this purpose.

Equation 31-160

$$v_i = \frac{v_{adj,i}}{N_i}$$

where

v_i = lane flow rate for lane group *i* expressed in through passenger cars per hour per lane (tpc/h/ln);

$v_{adj,i}$ = equivalent through movement flow rate for lane group i (tpc/h); and

N_i = number of lanes associated with lane group i, accounting for de facto lanes (ln).

Step 4: Determine Critical Lane Groups

The critical lane groups are identified and the sum of critical-lane flow rates is determined in this step. Critical lane groups represent the unique combination of conflicting lane groups that have the highest total flow rate. These critical lane groups dictate the amount of green time required during each phase. They also dictate the total cycle length required for the intersection. The critical lane groups for the north–south and east–west approaches are assessed independently.

This step consists of three tasks. During the first task, the right-turn flow rate is adjusted to account for right-turn capacity during the complementary left-turn phase. During the second task, the critical lane groups are identified. During the third task, the critical-lane group flow rates are added to determine the sum of critical-lane flow rates.

Step 4a. Adjust Right-Turn Flow Rate

There may be situations in which an exclusive right-turn lane could have a higher flow rate than the adjacent through lane(s). In this situation, the right turns that could occur simultaneously with a protected left-turn movement from the cross street should be deducted from the right-turn flow rate. For example, if the exclusive northbound right-turn flow rate is 300 tpc/h/ln and the protected westbound left-turn flow rate is 125 tpc/h/ln, 125 northbound right-turn vehicles should be assumed to depart the intersection during the westbound left-turn phase. Thus, 125 should be deducted from the total northbound right-turn flow rate, resulting in an adjusted northbound right-turn flow rate of 175 tpc/h/ln. This adjustment is only necessary when the right-turn lane group is critical. If that is the case, the rules described in Step 4b should replace the through lane group flow rate with the right-turn lane group flow rate.

Step 4b. Identify Critical Lane Groups

The lane groups that are determined to be critical are identified in this task. The rules for making this determination are dependent on the left-turn operational mode and phase sequence. Each of the combinations addressed by the planning-level analysis application is discussed in the following paragraphs.

Protected operation—with left-turn phase. When opposing approaches use protected left-turn operation, there are two possible lane group combinations that could determine the critical-lane flow rate. Each combination comprises a left-turn lane group and its opposing through (or right-turn) lane group. The flow rate for each lane group pair is added. The maximum of these two sums defines the critical-lane flow rate. For the east–west approaches, the critical-lane flow rate is computed with Equation 31-161.

Equation 31-161

$$V_{c,prot,1} = \max \begin{bmatrix} v_{EBlt} + v_{WBth} \\ v_{WBlt} + v_{EBth} \end{bmatrix}$$

where

$V_{c,prot,1}$ = critical-lane flow rate for protected left-turn operation on the east–west approaches (tpc/h/ln), and

v_i = lane flow rate for lane group i (i = *EBlt*: eastbound left turn, *WBlt*: westbound left turn, *EBth*: eastbound through, *WBth*: westbound through) (tpc/h/ln).

The two lane groups that add to produce the largest critical-lane flow rate in Equation 31-161 represent the critical lane groups for the east–west street.

Similarly, for north–south approaches with protected left-turn operation, the critical-lane flow rate is computed with Equation 31-162.

Equation 31-162

$$V_{c,prot,2} = \max \begin{bmatrix} v_{NBlt} + v_{SBth} \\ v_{SBlt} + v_{NBth} \end{bmatrix}$$

where

$V_{c,prot,2}$ = critical-lane flow rate for protected left-turn operation on the north–south approaches (tpc/h/ln), and

v_i = lane flow rate for lane group i (i = *NBlt*: northbound left turn, *SBlt*: southbound left turn, *NBth*: northbound through, *SBth*: southbound through) (tpc/h/ln).

The two lane groups that add to produce the largest critical-lane flow rate in Equation 31-162 represent the critical lane groups for the north–south street.

Permitted operation—no left-turn phase. When opposing approaches use permitted operation, the critical-lane flow rate will be the highest lane flow rate of all lane groups associated with the pair of approaches. For the east–west approaches, the critical-lane flow rate is computed with Equation 31-163.

Equation 31-163

$$V_{c,perm,1} = \max\,(v_{EBlt}, v_{EBth}, v_{EBrt}, v_{WBlt}, v_{WBth}, v_{WBrt})$$

where

$V_{c,perm,1}$ = critical-lane flow rate for permitted left-turn operation on the east–west approaches (tpc/h/ln), and

v_i = lane flow rate for lane group i (i = *EBlt*: eastbound left turn, *WBlt*: westbound left turn, *EBth*: eastbound through, *WBth*: westbound through, *EBrt*: eastbound right turn, *WBrt*: westbound right turn) (tpc/h/ln).

The lane group that produces the largest critical-lane flow rate in Equation 31-163 represents the critical lane group for the east–west street.

Similarly, for north–south approaches with permitted left-turn operation, the critical-lane flow rate is computed with Equation 31-164.

Equation 31-164

$$V_{c,perm,2} = \max\,(v_{SBlt}, v_{SBth}, v_{SBrt}, v_{NBlt}, v_{NBth}, v_{NBrt})$$

where

$V_{c,perm,2}$ = critical-lane flow rate for permitted left-turn operation on the north–south approaches (tpc/h/ln), and

v_i = lane flow rate for lane group i (i = $SBlt$: southbound left turn, $NBlt$: northbound left turn, $SBth$: southbound through, $NBth$: northbound through, $SBrt$: southbound right turn, $NBrt$: northbound right turn) (tpc/h/ln).

The lane group that produces the largest critical-lane flow rate in Equation 31-164 represents the critical lane group for the north–south street.

Protected operation—split phasing. When opposing approaches use split phasing (i.e., when only one approach is served during a phase), the critical-lane flow rate for a given approach will be the highest lane flow rate of all lane groups for that approach. The critical-lane flow rate for the two opposing approaches will be the sum of the highest lane flow rate for each approach. For the east–west approaches, the critical-lane flow rate is computed with Equation 31-165.

$$V_{c,\text{split},1} = \max\left(v_{EBlt}, v_{EBth}, v_{EBrt}\right) + \max\left(v_{WBlt}, v_{WBth}, v_{WBrt}\right)$$

Equation 31-165

where $V_{c,\text{split},1}$ is the critical-lane flow rate for split phasing on the east–west approaches (tpc/h/ln).

The two lane groups that add to produce the largest critical-lane flow rate in Equation 31-165 represent the critical lane groups for the east–west street.

Similarly, for the north–south approaches with split phasing, the critical-lane flow rate is computed with Equation 31-166.

$$V_{c,\text{split},2} = \max(v_{SBlt}, v_{SBth}, v_{SBrt}) + \max(v_{NBlt}, v_{NBth}, v_{NBrt})$$

Equation 31-166

where $V_{c,\text{split},2}$ is the critical-lane flow rate for split phasing on the north–south approaches (tpc/h/ln).

The two lane groups that add to produce the largest critical-lane flow rate in Equation 31-166 represent the critical lane groups for the north–south street.

Protected-permitted operation—with left-turn phase. If protected-permitted operation is to be evaluated, the analyst should refer to the supplemental procedure in the Protected-Permitted Left-Turn Operations subsection.

Step 4c. Calculate the Sum of Critical-Lane Flow Rates

Once the critical lane groups have been identified, the sum of critical-lane flow rates for the intersection can be computed by adding the lane flow rate associated with each critical lane group. Alternatively, the sum of critical-lane flow rates can be computed by adding the critical-lane group flow rate for each intersecting street, as calculated in the previous task. The following four cases illustrate this technique for some example combinations of left-turn operation and phase sequence using Equation 31-167 through Equation 31-170.

Case 1: East–west and north–south approaches use protected operation—with left-turn phase.

$$V_c = V_{c,prot,1} + V_{c,prot,2}$$

Equation 31-167

where V_c is the sum of the critical-lane flow rates (tpc/h/ln).

Case 2: East–west and north–south approaches use permitted operation—no left-turn phase.

Equation 31-168

$$V_c = V_{c,perm,1} + V_{c,perm,2}$$

Case 3: East–west approaches use protected operation—with left-turn phase and north–south approaches use permitted operation—no left-turn phase.

Equation 31-169

$$V_c = V_{c,prot,1} + V_{c,perm,2}$$

Case 4: East–west approaches use protected operation—with left-turn phase and north–south approaches use protected operation—split phasing.

Equation 31-170

$$V_c = V_{c,prot,1} + V_{c,split,2}$$

Step 4d. Identify Critical Phases

The critical phases identified in this task are used in Part II. If Part II is not part of the analysis, then this task can be skipped.

For this task, one critical phase is associated with each critical lane group, as identified in Step 4b. The flow rate that corresponds to a critical lane group (and critical phase i) is called the critical-lane flow rate $v_{c,i}$. By definition, the sum of these critical-lane flow rates equals the sum of critical-lane flow rates V_c.

For example, consider an intersection for which Equation 31-167 is determined to be applicable (i.e., the intersection has protected operation—with left-turn phase on both approaches). If the eastbound left-turn and westbound through phases are found to yield the critical-lane flow rate $V_{c,prot,1}$, then the eastbound left-turn phase and the westbound through phase are identified as critical phases. The critical-lane flow rates for the east–west approaches are $v_{c,EBlt}$ (= v_{EBlt}) and $v_{c,WBth}$ (= v_{WBth}).

Step 5: Determine Intersection Sufficiency

This step consists of four tasks. The first task is to determine the cycle length, and the second is to calculate intersection capacity. The third task is to compute the intersection volume-to-capacity ratio. The fourth task is to determine whether the intersection is operating under, near, or over its capacity.

If local data describing cycle length and base saturation flow rate are not available, then a default intersection capacity c_I of 1,650 tpc/h/ln can be used. This default value reflects a base saturation flow rate of 1,900 pc/h/ln, a lost time of 4.0 s per phase, and a cycle length equal to 30 s per critical phase. If the default intersection capacity is used, then the analyst can proceed to Step 5c.

Step 5a. Calculate Cycle Length

If cycle length is known, then the analyst can proceed to Step 5b.

For purposes of conducting a planning-level analysis, the analyst can assume a cycle length equal to 30 s for each critical phase. For example, an intersection with a protected left-turn phase for each of the eastbound and westbound approaches and permitted left-turn operation for the northbound and southbound approaches could be assumed to have a 90-s cycle length. The selection of a cycle length in practice should be based on consideration of multiple factors including (*a*) local agency policies and practices and (*b*) needs of nonmotorized users.

Step 5b. Calculate Intersection Capacity

Intersection capacity is calculated with Equation 31-171.

$$c_I = s_o \frac{C - (n_{cp}\, l_t)}{C}$$

Equation 31-171

where

c_I = intersection capacity (tpc/h/ln),

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

C = cycle length (s),

n_{cp} = number of critical phases, and

l_t = phase lost time (s).

A default phase lost time of 4.0 s for each critical phase is recommended. A default value for base saturation flow rate can be obtained from Exhibit 19-11.

Step 5c. Calculate the Intersection Volume-to-Capacity Ratio

The critical intersection volume-to-capacity ratio is calculated with Equation 31-172.

$$X_c = \frac{V_c}{c_I}$$

Equation 31-172

where

X_c = critical intersection volume-to-capacity ratio,

V_c = sum of critical-lane flow rates (tpc/h/ln), and

c_I = intersection capacity (tpc/h/ln).

Step 5d. Assess Intersection Sufficiency

The objective of this task is to assess the sufficiency of the intersection in terms of its ability to accommodate a given demand level. Exhibit 31-37 provides guidance for determining whether an intersection is operating under, near, or over its available capacity.

The analyst may stop at this point or may continue with Part II to determine delay and LOS.

Exhibit 31-37
Planning-Level Analysis: Intersection Volume-to-Capacity Ratio Assessment Levels

Critical Intersection Volume-to-Capacity Ratio	Description	Capacity Assessment
<0.85	All demand is able to be accommodated; delays are low to moderate.	Under
0.85–0.98	Demand for critical lane groups is near capacity and some lane groups require more than one cycle to clear the intersection; all demand is able to be processed within the analysis period; delays are moderate to high.	Near
>0.98	Demand for critical lane groups is just able to be accommodated within a cycle but often requires multiple cycles to clear the intersection; delays are high and queues are long.	Over

Part II: Delay and Level of Service

Part II builds on the results of Part I by allowing the user to calculate capacity, delay, and LOS.

Step 6: Calculate Capacity

This step consists of two tasks. For the first task, the analyst calculates the effective green time for each critical phase. For the second task, the analyst calculates the volume-to-capacity ratio for each lane group.

Step 6a. Calculate Effective Green Times

If the effective green time for each critical phase is known, then the analyst can proceed to Step 6b.

The total effective green time available for all critical phases is equal to the cycle length minus the total lost time per cycle. This calculation is shown in Equation 31-173.

Equation 31-173

$$g_{tot} = C - (n_{cp} l_t)$$

where

g_{tot} = total effective green time in the cycle (s),

C = cycle length (s),

n_{cp} = number of critical phases, and

l_t = phase lost time (s).

A default phase lost time of 4.0 s for each critical phase is recommended.

The total effective green time is allocated to each critical phase in proportion to the lane flow rate for each critical phase. Equation 31-174 is used to compute the effective green time for a given critical lane group.

Equation 31-174

$$g_{c,i} = g_{tot}\left(\frac{v_{c,i}}{V_c}\right)$$

where

$g_{c,i}$ = effective green time for critical lane group i (s),

g_{tot} = total effective green time in the cycle (s),

$v_{c,i}$ = lane flow rate for critical lane group i (tpc/h/ln), and

V_c = sum of the critical-lane flow rates (tpc/h/ln).

The effective green time for a noncritical lane group is set equal to the effective green time for its counterpart critical lane group that occurs concurrently during the same phase.

Finally, the effective green time g_i for each phase i is set equal to the effective green time that is computed for the corresponding lane group. The effective green time computed in this manner should be reviewed against policy requirements and other considerations (such as the minimum green time based on driver expectancy and the time required for pedestrians to cross the approach).

Step 6b. Calculate Capacity and Volume-to-Capacity Ratios

The lane group capacity and volume-to-capacity ratio can be computed with Equation 31-175 and Equation 31-176, respectively.

$$c_i = s_o \, N_i \, \frac{g_i}{C}$$ Equation 31-175

$$X_i = \frac{N_i \, v_i}{c_i}$$ Equation 31-176

where

c_i = capacity of lane group i (tpc/h);

g_i = effective green time for lane group i (s);

N_i = number of lanes associated with lane group i, accounting for de facto lanes (ln);

X_i = volume-to-capacity ratio for lane group i;

v_i = lane flow rate for lane group i (tpc/h/ln); and

C = cycle length (s).

The capacity for each lane group is based on the base saturation flow rate s_o. A default value for base saturation flow rate can be obtained from Exhibit 19-11. This rate is not adjusted for parking activity, heavy vehicles, and so forth because these adjustments are applied in Step 2 to the lane group flow rate.

Equation 31-177 and Equation 31-178 can be used to compute the intersection capacity and intersection volume-to-capacity ratio, respectively.

$$c_{\text{sum}} = s_o \, \frac{\sum_{i=1}^{n_{cp}} g_{c,i}}{C}$$ Equation 31-177

$$X_c = \frac{V_c}{c_{\text{sum}}}$$ Equation 31-178

where c_{sum} is the intersection capacity (tpc/h/ln).

Step 7: Determine Delay and Level of Service

The control delay for each lane group is calculated by using Equation 31-179 with Equation 31-180 and Equation 31-181.

Equation 31-179

$$d_i = d_{1,i} + d_{2,i}$$

with

Equation 31-180

$$d_{1,i} = PF_i \frac{0.5\, C(1 - g_i/C)^2}{1 - [\min(1, X_i)\, g_i/C]}$$

Equation 31-181

$$d_{2,i} = 225\left[(X_i - 1) + \sqrt{(X_i - 1)^2 + \frac{16\, X_i}{c_i}}\right]$$

where

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

$d_{1,i}$ = uniform delay for lane group *i* (s/veh),

$d_{2,i}$ = incremental delay for lane group *i* (s/veh),

PF_i = progression adjustment factor for lane group *i*, and

all other variables are as previously defined.

The progression adjustment factor describes the arrival distribution for the subject lane group, which may be influenced by an upstream traffic signal. Recommended progression adjustment factors are shown in Exhibit 31-38.

Exhibit 31-38
Planning-Level Analysis: Progression Adjustment Factor

Quality of Progression	Conditions That Describe Arrivals Associated with the Subject Lane Group	Progression Factor *PF*
Good progression	(a) Vehicles arrive in platoons during the green interval, OR (b) most vehicles arrive during the green interval.	0.70
Random arrivals (default)	(a) The phase serving the subject lane group is not coordinated with the upstream traffic signal, OR (b) the intersection is sufficiently distant from other signalized intersections as to be considered isolated.	1.00
Poor progression	(a) Vehicles arrive in platoons during the red interval, OR (b) most vehicles arrive during the red indication.	1.25

Lane group delay may be aggregated for each approach and for the intersection as a whole. The aggregation process is the same as that in the motorized vehicle methodology in Chapter 19 using Equation 19-28 and Equation 19-29.

Delay values may be compared with the criteria in Exhibit 19-8 to determine the LOS for a lane group, approach, or the intersection as a whole.

Protected-Permitted Left-Turn Operations

The procedure described in this subsection applies to the analysis of protected-permitted left-turn operation. The effective green time is a required input data item. If it is known or can be estimated, then the supplemental guidance in this subsection can be used with the planning-level analysis application.

Step 2: Convert Movement Volumes to Through Passenger-Car Equivalents

The guidance provided in this subsection supplements that provided in Step 2 of the planning-level analysis application. The objective is to compute an equivalency factor for protected-permitted left-turn operation that reflects the left-turn vehicle's overall effect on operations.

A single left-turn equivalency factor is computed for both the protected and the permitted time periods. Exhibit 31-33 is used to identify the equivalency factor for protected left-turn operation during the left-turn phase. It is also used to identify the equivalency factor for permitted left-turn operation during the through phase. A single factor is calculated that weighs these two equivalency factors in proportion to the effective green times of each time period. Equation 31-182 is used to compute the single equivalency factor for left turns.

$$E_{LT} = \frac{E_{LT,pt}\, g_{lt,pt} + E_{LT,pm}\, g_{lt,pm}}{g_{lt,pt} + g_{lt,pm}}$$

Equation 31-182

where

E_{LT} = equivalency factor for left turns,

$E_{LT,pt}$ = equivalency factor for protected left-turn operation,

$E_{LT,pm}$ = equivalency factor for permitted left-turn operation,

$g_{lt,pt}$ = effective green time for the protected left-turn phase (s), and

$g_{lt,pm}$ = effective green time for permitted left-turn operation during the through phase (s).

The equivalency factor computed with Equation 31-182 is used in Equation 31-157 to compute the equivalent through movement flow rate for the left-turn lane group. The effective green time for the first time period of the protected-permitted operation includes the yellow interval that occurs between the two periods.

Step 4: Determine Critical Lane Groups

The guidance provided in this subsection supplements that provided in Step 4 of the planning-level analysis application. The objective is to compute the left-turn lane flow rate during the protected left-turn phase and then use this value to identify the critical lane groups.

The equivalent through-car flow rate in the left lane during the protected left-turn phase is estimated by distributing the lane flow rate for the left-turn lane group proportionally among the protected and permitted periods. The flow rate for the protected left-turn period is computed with Equation 31-183.

$$v_{lt,pt} = v_{lt}\frac{g_{lt,pt}}{g_{lt,pt} + g_{lt,pm}}$$

Equation 31-183

where

$v_{lt,pt}$ = lane flow rate for the left-turn lane group during the protected left-turn phase (tpc/h/ln), and

v_{lt} = lane flow rate for the left-turn lane group (tpc/h/ln).

In the process of identifying the critical lane groups (and related flow rate), only the lane flow rate during the protected left-turn phase $v_{lt,pt}$ is used for the left-turn lane group. The critical-lane flow rate is then determined by using the rules described for protected operation–with left-turn phase in Step 4b above.

The remainder of the planning-level analysis application does not change. In Step 7, the lane flow rate for the left-turn lane group v_{lt} is used to determine the delay and LOS.

WORKSHEETS

This subsection includes a series of worksheets that can be used to document an application of the planning-level analysis application. These worksheets are as follows:

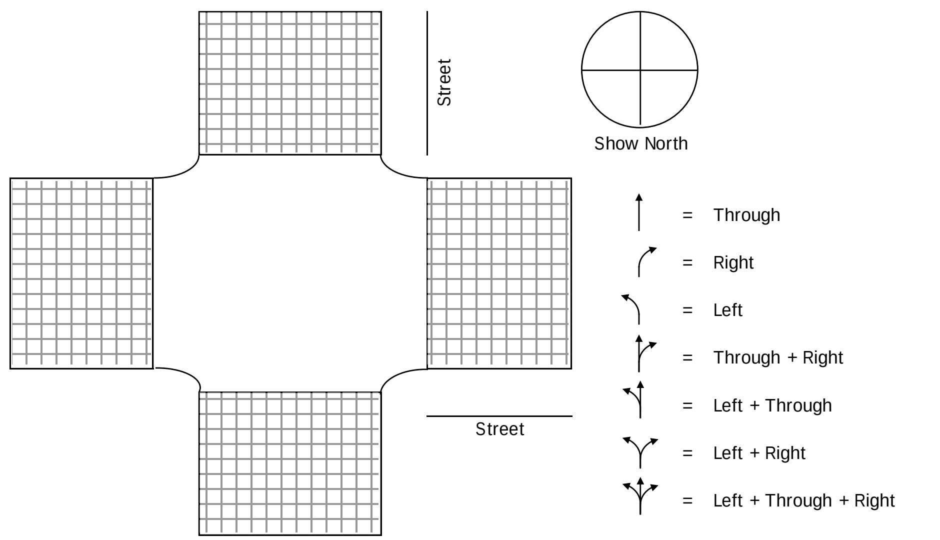
- Input Worksheet (Exhibit 31-39),
- Left-Turn Treatment Worksheet (Exhibit 31-40),
- Intersection Sufficiency Worksheet (Exhibit 31-41), and
- Delay and LOS Worksheet (Exhibit 31-42).

Exhibit 31-39
Planning-Level Analysis: Input Worksheet

PLANNING-LEVEL ANALYSIS: INPUT WORKSHEET

General Information		Site Information	
Analyst	________	Intersection	________
Agency or Company	________		
Date Performed	________	Jurisdiction	________
Analysis Time Period	________	Analysis Year	________

Intersection Geometry



Volume and Signal Input

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Required Data												
Volume (veh/h)												
Number of lanes												
Lane use (exclusive or shared)												
Optional Data[1]												
Heavy vehicles (%)												
On-street parking presence (no, yes)												
Pedestrian activity (none, low, med., high, extreme)												
Left-turn operation and phase sequence[2]												
Effective green time (s)[3,4]												
Progression quality (good, random, poor)[4]												

Peak hour factor ________ Cycle length (s) ________ Base saturation flow rate (pc/h/ln) ________

Notes

1. Optional input data (guidance is provided for estimating these data if they are not known).
2. Combinations addressed: (a) protected operation—with left-turn phase, (b) permitted operation—no left-turn phase, (c) protected operation—split phasing, (d) protected-permitted operation—with left-turn phase
3. Data required for Part I analysis if "protected-permitted operation—with left-turn phase" is present.
4. Data required for Part II analysis.

Exhibit 31-40
Planning-Level Analysis: Left-Turn Treatment Worksheet

PLANNING-LEVEL ANALYSIS: LEFT-TURN TREATMENT WORKSHEET

General Information

Description ______________________________

Check # 1. Left-Turn Lane Check

Approach	EB	WB	NB	SB
Number of left-turn lanes				
Protected left turn (Y or N)?				

If the number of left-turn lanes on any approach exceeds 1, then it is recommended that the left turns on that the approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check # 2. Minimum Volume Check

Approach	EB	WB	NB	SB
Left-turn volume				
Protected left turn (Y or N)?				

If left-turn volume on any approach exceeds 240 veh/h, then it is recommended that the left turns on that the approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Check # 3. Minimum Cross-Product Check

Approach	EB	WB	NB	SB
Left-turn volume, V_L (veh/h)				
Opposing mainline volume, V_o (veh/h)				
Cross product (V_L * V_o)				
Opposing through lanes				
Protected left turn (Y or N)?				

Minimum Cross-Product Values for Recommending Left-Turn Protection

Number of Through Lanes	Minimum Cross Product
1	50,000
2	90,000
3	110,000

If the cross product on any approach exceeds the above values, then it is recommended that the left turns on that approach be protected. Those approaches with protected left turns need not be evaluated in subsequent checks.

Notes

1. If any approach is recommended for left-turn protection but the analyst evaluates it as having permitted operation, then the planning-level analysis method may give overly optimistic results. The analyst should instead use the automobile methodology described in Chapter 19, Signalized Intersections.
2. All volumes used in this worksheet are unadjusted hourly volumes.

Exhibit 31-41
Planning Level Analysis: Intersection Sufficiency Worksheet

PLANNING-LEVEL ANALYSIS: INTERSECTION SUFFICIENCY WORKSHEET						
General Information						
Description:						
East-West Approaches						
	Eastbound			Westbound		
	Left	Through	Right	Left	Through	Right
Movement volume, V (veh/h)						
Equivalency factor for heavy vehicles, E_{hv}						
Equivalency factor for peaking char., E_{PHF}						
Equivalency factor for right turns, E_{RT}						
Equivalency factor for left turns, E_{LT}[1]						
Equivalency factor for parking activity, E_p						
Equivalency factor for lane utilization, E_{LU}						
Equivalency factor for other conditions, E_{other}						
Equivalent through mvmt. flow rate (tpc/h) $v_{adj} = V\,E_{HV}\;E_{PHF}\;E_{LT}\;E_{RT}\;E_p\;E_{LU}\;E_{\text{other}}$						
Number of lanes, N						
Lane flow rate, v (tpc/h/ln) $v = v_{adj} / N$						
Critical lane flow rate, V_c (tpc/h/ln)						
Critical lane group (indicate with "X")						
Critical lane group flow rate, v_c (tpc/h/ln)						
Supplemental Calculations for Protected-Permitted Operation						
Equivalency factor for prot. left turn, $E_{LT,pt}$						
Equivalency factor for perm. left turn, $E_{LT,pm}$						
Effective green for prot. left turn, $g_{lt,pt}$ (s)						
Effective green for perm. left turn, $g_{lt,pm}$ (s)						
Equivalency factor for left turns, E_{LT} $E_{LT} = (E_{LT,pt}\;g_{lt,pt} + E_{LT,pm}\;g_{lt,pm}) / (g_{lt,pt} + g_{lt,pm})$						
North-South Approaches						
	Northbound			Southbound		
	Left	Through	Right	Left	Through	Right
Movement volume, V (veh/h)						
Equivalency factor for heavy vehicles, E_{hv}						
Equivalency factor for peaking char., E_{PHF}						
Equivalency factor for right turns, E_{RT}						
Equivalency factor for left turns, E_{LT}[1]						
Equivalency factor for parking activity, E_p						
Equivalency factor for lane utilization, E_{LU}						
Equivalency factor for other conditions, E_{other}						
Equivalent through mvmt. flow rate (tpc/h) $v_{adj} = V\,E_{HV}\;E_{PHF}\;E_{LT}\;E_{RT}\;E_p\;E_{LU}\;E_{\text{other}}$						
Number of lanes, N						
Lane flow rate, v (tpc/h/ln) $v = v_{adj} / N$						
Critical lane flow rate, V_c (tpc/h/ln)						
Critical lane group (indicate with "X")						
Critical lane group flow rate, v_c (tpc/h/ln)						
Supplemental Calculations for Protected-Permitted Operation						
Equivalency factor for prot. left turn, $E_{LT,pt}$						
Equivalency factor for perm. left turn, $E_{LT,pm}$						
Effective green for prot. left turn, $g_{lt,pt}$ (s)						
Effective green for perm. left turn, $g_{lt,pm}$ (s)						
Equivalency factor for left turns, E_{LT} $E_{LT} = (E_{LT,pt}\;g_{lt,pt} + E_{LT,pm}\;g_{lt,pm}) / (g_{lt,pt} + g_{lt,pm})$						

Intersection Sufficiency Assessment			
Number of critical phases, n_{cp}		Intersection capacity, c_I (tpc/h/ln) $c_I = s_o\;[C - (n_{cp}\;4.0)]/C$	
Sum of critical lane flow rates, V_c (tpc/h/ln)		Critical intersection vol.-to-capacity ratio, X_c $X_c = V_c\;/\;c_I$	
Intersection status (relationship to capacity)	Under ___ Near ___ Over ___		
Note			
1. If the approach has protected-permitted operation, use the supplemental calculations section to compute E_{LT}.			

Exhibit 31-42
Planning-Level Analysis: Delay and LOS Worksheet

PLANNING-LEVEL ANALYSIS: DELAY AND LOS WORKSHEET

General Information

Description ____

Green Time Calculation

Total effective green time, g_{tot} (s) $g_{tot} = C - (n_{cp}\ 4.0)$	

East-West Approaches	Eastbound			Westbound		
	Left	Through	Right	Left	Through	Right
Critical lane group flow rate, v_c (tpc/h/ln)[1]						
Effective green time for critical lane group, g_c (s) $g_c = g_{tot}\ v_c / V_c$						
	Phase No. 1		Phase No. 2		Phase No. 3	
Effective green time, g (s)						

North-South Approaches	Northbound			Southbound		
	Left	Through	Right	Left	Through	Right
Critical lane group flow rate, v_c (tpc/h/ln)[1]						
Effective green time for critical lane group, g_c (s) $g_c = g_{tot}\ v_c / V_c$						
	Phase No. 1		Phase No. 2		Phase No. 3	
Effective green time, g (s)						

Control Delay and LOS

	EB			WB			NB			SB		
Lane group												
Effective green time, g (s)												
Green-to-cycle-length ratio, g/C												
Number of lanes, N[1]												
Lane group capacity, c (veh/h) c = 1900 N g/C												
Lane flow rate, v (tpc/h/ln)[1]												
Volume-to-capacity ratio, X X = (N v)/c												
Progression adjustment factor, PF												
Uniform delay, d_1 (s/veh)												
Incremental delay, d_2 (s/veh)												
Control delay, $d = d_1 + d_2$ (s/veh)												
Approach delay, d_A (s/veh) $d_A = \Sigma(d\ N\ v)/\Sigma(N\ v)$												
Approach flow rate, V_A (veh/h)												
Intersection delay, d_I (s/veh) $d_I = \Sigma(d_A\ V_A)/\Sigma V_A$				Intersection LOS (Exhibit 19-8)								
Intersection capacity, c_{sum} (tpc/h/ln) $c_{sum} = 1900\ (\Sigma g_c) / C$			Critical intersection vol.-to-capacity ratio, X_c $X_c = V_c / c_{sum}$									

Notes

1. Value obtained from the Intersection Sufficiency Worksheet.

6. FIELD MEASUREMENT TECHNIQUES

This section describes two techniques for estimating key traffic characteristics by using field data. The first subsection describes a technique for estimating control delay. The second subsection describes a technique for estimating saturation flow rate.

FIELD MEASUREMENT OF INTERSECTION CONTROL DELAY

Delay can be measured at existing intersections as an alternative to estimating delay by using the motorized vehicle methodology in Chapter 19, Signalized Intersections. Various techniques can be used for measuring delay, including a test-car survey, vehicle path tracing, input–output analysis, and queue counting. The first three techniques tend to require more time to implement than the last technique, but they provide more accurate delay estimates. They are often limited to sampling when implemented manually. They may be more appropriate when oversaturated conditions are present. The first two techniques can be used to estimate delay on either a movement basis or a lane group basis. The last two techniques are more amenable to delay measurement on a lane group basis.

The queue-count technique is recommended for control delay measurement. It is based on direct observation of vehicle-in-queue counts for a subject lane group. It normally requires two field personnel for each lane group surveyed. Also needed are (*a*) a multifunction digital watch that includes a countdown-repeat timer, with the countdown interval in seconds; and (*b*) a volume-count board with at least two tally counters. Alternatively, a laptop computer can be programmed to emit audio count markers at user-selected intervals, take volume counts, and execute real-time delay computations.

The queue-count technique is applicable to all undersaturated lane groups. Significant queue buildup can make the technique impractical for oversaturated lane groups or lane groups with limited storage length. If queues are lengthy, then the technique should be modified by subdividing the lane group into manageable segments (or zones) and assigning an observer to each zone. Each observer then counts queued vehicles in his or her assigned zone.

If queues are lengthy or the volume-to-capacity ratio is near 1.0, then care must be taken to continue the vehicle-in-queue count past the end of the arrival count period, as detailed in subsequent paragraphs. This extended counting period is required for consistency with the analytic delay equation used in the chapter text.

The queue-count technique does not directly measure delay during deceleration and during a portion of acceleration. These delay elements are very difficult to measure without sophisticated tracking equipment. Nevertheless, this technique has been shown to yield a reasonable estimate of control delay by application of appropriate adjustment factors (*8*, *9*). One adjustment factor accounts for sampling errors that may occur. Another factor accounts for unmeasured acceleration–deceleration delay. This adjustment factor is a function of the number of vehicles in queue each cycle and the approach speed.

Approach Speed

Exhibit 31-43 shows a worksheet that can be used for recording observations and computing control delay for the subject lane group. Before starting the survey, observers need to estimate the average approach speed during the study period. Approach speed is the speed at which vehicles would pass unimpeded through the intersection if the signal were green for an extended period and volume was light. This speed may be obtained by driving through the intersection a few times when the signal is green and there is no queue. The approach speed is recorded at an upstream location that is least affected by the operation of the subject signalized intersection as well as the operation of any other signalized intersection.

Survey Period

The duration of the survey period must be clearly defined in advance so the last arriving vehicle or vehicles that stop in the period can be identified and counted until they exit the intersection. It is logical to define the survey period on the basis of the same considerations used to define an evaluation analysis period (as described in Section 3 of Chapter 19). A typical survey period is 15 min.

Count Interval

The survey technique is based on recording a vehicle-in-queue count at specific points in time. A count interval in the range of 10 to 20 s has been found to provide a good balance between delay estimate precision and observer capability. The actual count interval selected from this range is based on consideration of survey period duration and the type of control used at the intersection.

The count interval *should* be an integral divisor of the survey period duration. This characteristic ensures that a complete count of events is taken for the full survey period. It also allows easier coordination of observer tasks during the field study. For example, if the study period is 15 min, the count interval can be 10, 12, 15, 18, or 20 s.

If the intersection has pretimed or coordinated-actuated control, the count interval *should not* be an integral divisor of the cycle length. This characteristic eliminates potential survey bias due to queue buildup in a cyclical pattern. For example, if the cycle length is 120 s, the count interval can be 11, 13, 14, 16, 17, 18, or 19 s.

If the intersection has actuated control, the count interval may be chosen as the most convenient value for conducting the field survey with consideration of survey period duration.

Exhibit 31-43
Control Delay Field Study Worksheet

INTERSECTION CONTROL DELAY WORKSHEET			
General Information		*Site Information*	
Analyst		Intersection	
Agency or Company		Area Type	❑ CBD ❑ Other
Date Performed		Jurisdiction	
Analysis Time Period		Analysis Year	
Input Initial Parameters			
Number of lanes, N		Total vehicles arriving, V_{tot}	
Approach speed, S_a (mi/h)		Stopped-vehicle count, V_{stop}	
Survey count interval, I_s (s)		Cycle length, C (s)	

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue, Count Interval 1	2	3	4	5	6	7	8	9	10
Total											

Computations

Total vehicles in queue, ΣV_{iq}	veh	Number of cycles surveyed, N_c	
Time-in-queue per vehicle, $d_{vq} = \left(I_s \dfrac{\Sigma V_{iq}}{V_{tot}}\right) 0.9$	s/veh	Fraction of vehicles stopping, $FVS = \dfrac{V_{stop}}{V_{tot}}$	
No. of vehicles stopping/lane/cycle = $\dfrac{V_{stop}}{(N_c \times N)}$	veh/ln	Accel-decel correction delay, $d_{ad} = FVS \times CF$	s/veh
Acceleration-deceleration correction factor, CF	s/veh	Control delay, $d = d_{vq} + d_{ad}$	s/veh

Measurement Technique

The survey should begin at the start of the red indication associated with the subject lane group. Two observers are required for data collection. The duties of each observer are described in the following paragraphs.

Observer 1 Tasks

1. Observer 1 keeps track of the end of the standing queue in each lane of the subject lane group. For purposes of the survey, a vehicle is considered as having joined the queue when it approaches within one car length of a

stopped vehicle and is itself about to stop. This definition is used because of the difficulty of keeping track of the moment when a vehicle comes to a stop.

2. At the start of each count interval, Observer 1 records the number of vehicles in queue in all lanes of the subject lane group. The countdown-repeat timer on a digital watch can be used to signal the count time. This count includes vehicles that arrive when the signal is actually green but stop because queued vehicles ahead have not yet started moving. All vehicles that join a queue are included in the vehicle-in-queue count until they "exit" the intersection. A through vehicle exits the intersection when its rear axle crosses the stop line. A turning vehicle exits the intersection the instant it clears the opposing through traffic (or pedestrians to which it must yield) and begins accelerating back to the approach speed. The vehicle-in-queue count often includes some vehicles that have regained speed but have not yet exited the intersection.
3. Observer 1 records the vehicle-in-queue count in the appropriate count-interval box on the worksheet. Ten boxes are provided for each "count cycle" (note that a count cycle is not the same as a signal cycle). Any number of boxes can be used to define the count cycle; however, as many as possible should be used to ensure best use of worksheet space. The clock time at the start of the count cycle is recorded in the first (far-left) column. The count cycle number is recorded in the second column of the sheet.
4. At the end of the survey period, Observer 1 continues taking vehicle-in-queue counts for all vehicles that arrived during the survey period until all of them have exited the intersection. This step requires the observer to make a mental note of the last stopping vehicle that arrived during the survey period in each lane of the lane group and continue the vehicle-in-queue counts until the last stopping vehicle or vehicles, plus all vehicles in front of the last stopping vehicle(s), exit the intersection. Stopping vehicles that arrive after the end of the survey period are not included in the final vehicle-in-queue counts.

Observer 2 Tasks

5. Observer 2 maintains three counts during the survey period. The first is a count of the vehicles that arrive during the survey period. The second is a count of the vehicles that arrive during the survey period and that stop one or more times. A vehicle stopping multiple times is counted only once as a stopping vehicle. The third count is the count of signal cycles, as measured by the number of times the red indication is presented for the subject lane group. For lane groups with a turn movement and protected or protected-permitted operation, the protected red indication is used for this purpose. If the survey period does not start or end at the same time as the presentation of a red indication, then the number of count intervals that occur in the interim can be used to estimate the fraction of the cycle that occurred at the start or end of the survey period.
6. Observer 2 enters all counts in the appropriate boxes on the worksheet.

Data Reduction Tasks

7. Sum each column of vehicle-in-queue counts, then sum the column totals for the entire survey period.
8. A vehicle recorded as part of a vehicle-in-queue count is assumed to be in queue, on average, for the time interval between counts. On this basis, the average time in queue per vehicle arriving during the survey period is estimated with Equation 31-184.

$$d_{vq} = 0.9\left(I_s \frac{\sum V_{iq}}{V_{tot}}\right)$$

Equation 31-184

where

d_{vq} = time in queue per vehicle (s/veh),

I_s = interval between vehicle-in-queue counts (s),

$\sum V_{iq}$ = sum of vehicle-in-queue counts (veh), and

V_{tot} = total number of vehicles arriving during the survey period (veh).

The 0.9 adjustment factor in Equation 31-184 accounts for the errors that may occur when the queue-count technique is used to estimate delay. Research has shown the adjustment factor value is fairly constant for a variety of conditions (*8*).

9. Compute the fraction of vehicles stopping and the average number of vehicles stopping per lane in each signal cycle, as indicated on the worksheet.
10. Use Exhibit 31-44 to look up the correction factor appropriate to the lane group approach speed and the average number of vehicles stopping per lane in each cycle. This factor adjusts for deceleration and acceleration delay, which cannot be measured directly with manual techniques (*9*).

Exhibit 31-44 Acceleration–Deceleration Correction Factor

Approach Speed (mi/h)	Acceleration–Deceleration Correction Factor *CF* (s/veh) As a Function of the Average Number of Vehicles Stopping		
	≤7 veh/ln/cycle	8–19 veh/ln/cycle	20–30 veh/ln/cycle[a]
≤37	+5	+2	–1
>37–45	+7	+4	+2
>45	+9	+7	+5

Note: [a] Vehicle-in-queue counts in excess of about 30 veh/ln/cycle are typically unreliable.

11. Multiply the correction factor by the fraction of vehicles stopping. Add this product to the time-in-queue value from Task 2 to obtain the estimate of control delay for the subject lane group.

Example Application

Exhibit 31-45 presents sample data for a lane group during a 15-min survey period. The intersection has a 115-s cycle. A 15-s count interval is selected because 15 is not an integral divisor of the cycle length, but it is an integral divisor of the survey period.

Exhibit 31-45
Example Control Delay Field Study Worksheet

INTERSECTION CONTROL DELAY WORKSHEET

General Information		*Site Information*	
Analyst		Intersection	*Cicero & Belmont*
Agency or Company		Area Type	☐ CBD ☒ Other
Date Performed		Jurisdiction	
Analysis Time Period	*PM*	Analysis Year	*2015*

Input Initial Parameters

Number of lanes, N	*2*	Total vehicles arriving, V_{tot}	*530*
Approach speed, S_a (mi/h)	*40*	Stopped-vehicle count, V_{stop}	*223*
Survey count interval, I_s (s)	*15*	Cycle length, C (s)	

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue — Count Interval 1	2	3	4	5	6	7	8	9	10
4:34	*1*	*3*	*8*	*11*	*15*	*12*	*2*	*0*	*2*		
	2	*6*	*12*	*15*	*16*	*6*	*0*	*0*	*2*		
	3	*7*	*11*	*14*	*14*	*2*	*0*	*0*			
	4	*5*	*7*	*10*	*13*	*13*	*2*	*0*	*1*		
4:42	*5*	*4*	*6*	*10*	*12*	*3*	*0*	*0*	*1*		
	6	*5*	*7*	*9*	*13*	*4*	*0*	*0*			
	7	*3*	*6*	*8*	*12*	*12*	*0*	*0*	*0*		
4:47	*8*	*4*	*7*	*11*	*16*	*9*	*0*				
Total		*37*	*64*	*88*	*111*	*61*	*4*	*0*	*6*		

Computations

Total vehicles in queue, Σv_{iq}	*371*	veh	Number of cycles surveyed, N_c	*7.8*	
Time-in-queue per vehicle, $d_{vq} = \left(I_s \frac{\Sigma V_{iq}}{V_{tot}}\right) 0.9$	*9.5*	s/veh	Fraction of vehicles stopping, $FVS = \frac{V_{stop}}{V_{tot}}$	*0.42*	
No. of vehicles stopping/lane/cycle $= \frac{V_{stop}}{(N_c \times N)}$	*14*	veh/ln	Accel-decel correction delay, $d_{ad} = FVS \times CF$	*1.7*	s/veh
Acceleration-deceleration correction factor, CF	*4*	s/veh	Control delay, $d = d_{vq} + d_{ad}$	*11.2*	s/veh

Exhibit 31-45 shows data are recorded for six, seven, or eight intervals during each count cycle. This choice is arbitrary and based solely on best use of worksheet space.

The data reduction results are shown at the bottom of the exhibit. A control delay of 11.2 s/veh is estimated for the subject lane group.

Exhibit 31-46 shows how the worksheet shown in Exhibit 31-45 would have been completed if a queue had remained at the end of the 15-min survey period. Only the vehicles that arrived during the 15-min period would be counted.

Exhibit 31-46
Example Worksheet with Residual Queue at End

INTERSECTION CONTROL DELAY WORKSHEET

General Information		*Site Information*	
Analyst		Intersection	*Cicero & Belmont*
Agency or Company		Area Type	☐ CBD ☒ Other
Date Performed		Jurisdiction	
Analysis Time Period	*PM*	Analysis Year	*1999*

Input Initial Parameters

Number of lanes, N	*2*	Total vehicles arriving, V_{tot}	
Approach speed, S_a (mi/h)	*40*	Stopped-vehicle count, V_{stop}	
Survey count interval, I_s (s)	*15*	Cycle length, C (s)	

Input Field Data

Clock Time	Cycle Number	Number of Vehicles in Queue — Count Interval 1	2	3	4	5	6	7	8	9	10
4:47	*8*	*4*	*7*	*11*	*16*	*9*	*0*	*Queue count in previous example*			
4:47	*8*	*4*	*4**	*4**	*4**	*0*	*First four in queue have cleared by now*				
			say 15-min. survey period ends here								
			4 - last stopping vehicles in survey period; count only until they clear.*								
Total		*37*	*61*	*81*	*99*	*52*	*4*	*0*	*6*		

Computations

Total vehicles in queue, ΣV_{iq} =	veh	Number of cycles surveyed, N_c =	
Time-in-queue per vehicle, $d_{vq} = \left(I_s \frac{\Sigma V_{iq}}{V_{tot}}\right) 0.9$	s/veh	Fraction of vehicles stopping, FVS = $\frac{V_{stop}}{V_{tot}}$	
No. of vehicles stopping/lane/cycle = $\frac{V_{stop}}{(N_c \times N)}$	veh/ln	Accel-decel correction delay, d_{ad} = FVS * CF	s/veh
Accel-decel correction factor, CF	s/veh	Control delay, $d = d_{vq} + d_{ad}$	s/veh

FIELD MEASUREMENT OF SATURATION FLOW RATE

This subsection describes a technique for quantifying the base saturation flow rate for local conditions. It provides a means of calibrating the saturation flow rate calculation procedure (described in Section 3 of Chapter 19) to reflect driver behavior at a local level. The technique is based on a comparison of field-measured saturation flow rate with the calculated saturation flow rate for a common set of lane groups at intersections in a given area.

Concepts

The saturation flow rate represents the maximum rate of flow in a traffic lane, as measured at the stop line during the green indication. It is usually achieved after 10 to 14 s of green, which corresponds to the front axle of the fourth to sixth queued passenger car crossing the stop line.

The base saturation flow rate represents the saturation flow rate for a traffic lane that is 12 ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. It is usually stable over a period of time in a given area and normally exhibits a relatively narrow distribution among intersections in that area.

The prevailing saturation flow rate is the rate measured in the field for a specific lane group at a specific intersection. It may vary significantly among intersections with similar lane groups because of differences in lane width, traffic composition (i.e., percentage of heavy vehicles), grade, parking, bus stops, lane use, and turning vehicle operation. If the intersections are located in different areas, then the prevailing saturation flow rate may also vary because of areawide differences in the base saturation flow rate.

The adjusted saturation flow rate is the rate computed by the procedure described in Chapter 19. It represents an estimate of the prevailing saturation flow rate. It can vary among intersections for the same reasons as stated above for the prevailing saturation flow rate. Any potential bias in the estimate is minimized by local calibration of the base saturation flow rate.

The prevailing saturation flow rate and the adjusted saturation flow rate are both expressed in units of vehicles. As a result, their value reflects the traffic composition in the subject traffic lane. In contrast, the base saturation flow rate is expressed in units of passenger cars and does not reflect traffic composition.

Measurement Technique

This subsection describes the technique for measuring the prevailing saturation flow rate for a given traffic lane. In general, vehicles are recorded when their front axles cross the stop line. The measurement period starts at the beginning of the green interval or when the front axle of the first vehicle in the queue passes the stop line. Saturation flow rate is calculated only from the data recorded after the fourth vehicle in the queue passes the stop line.

The vehicle’s front axle, the stop line, and the time the fourth queued vehicle crosses the stop line represent three key reference points for saturation flow measurement. These three reference points must be maintained to ensure consistency with the procedure described in Chapter 19 and to facilitate comparability of results with other studies. The use of other reference points on the vehicle, on the road, or in time may yield different saturation flow rates.

If the stop line is not visible or if vehicles consistently stop beyond the stop line, then an alternative reference line must be established. This reference line should be established just beyond the typical stopping position of the first queued vehicle. Vehicles should consistently stop behind this line. Observation of several cycles before the start of the study should be sufficient to identify this substitute reference line.

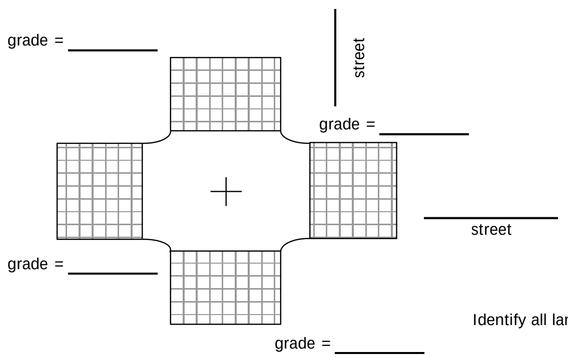
The following paragraphs describe the tasks associated with a single-lane saturation flow survey. A two-person field crew is recommended. However, one person with a tape recorder, push-button event recorder, or a notebook computer with appropriate software will suffice. The field notes and tasks identified in the following paragraphs must be adjusted according to the type of equipment used. A sample field worksheet for recording observations is included as Exhibit 31-47.

Exhibit 31-47
Saturation Flow Rate Field Study Worksheet

FIELD SATURATION FLOW RATE STUDY WORKSHEET

General Information		*Site Information*	
Analyst	____________	Intersection	____________
Agency or Company	____________	Area Type	❑ CBD ❑ Other
Date Performed	____________	Jurisdiction	____________
Analysis Time Period	____________	Analysis Year	____________

Lane Movement Input



Movements Allowed
❑ Through
❑ Right turn
❑ Left turn

Identify all lane movements and the lane studied

Input Field Measurement

	Cycle 1			Cycle 2			Cycle 3			Cycle 4			Cycle 5			Cycle 6		
Veh. in queue	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T	Time	HV	T
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
16																		
17																		
18																		
19																		
20																		
End of saturation																		
End of green																		
No. veh. > 20																		
No. veh. on yellow																		

Glossary and Notes

HV = Heavy vehicles (vehicles with more than 4 tires on pavement)
T = Turning vehicles (L = Left, R = Right)
Pedestrians and buses that block vehicles should be noted with the time that they block traffic, for example,
P12 = Pedestrians blocked traffic for 12 s
B15 = Bus blocked for 15 s

General Tasks

Measure and record the area type as well as the width and grade of the lane being studied. Enter these data in the lane movement input section of the field worksheet.

Select an observation point where the roadway reference line (e.g., stop line) for the surveyed lane and the corresponding signal heads are clearly visible. When a vehicle crosses this line unimpeded, it has entered the intersection conflict space for the purpose of saturation flow measurement. Left- or right-turning vehicles yielding to opposing through traffic or yielding to pedestrians are not recorded until they proceed through the opposing traffic or pedestrians.

Recorder Tasks

During the measurement period, note the last vehicle in the stopped queue when the signal turns green. Describe the last vehicle to the timer. Note on the worksheet which vehicles are heavy vehicles and which vehicles turn left or right. Record the time called out by the timer.

Timer Tasks

Start the stopwatch at the beginning of the green indication and notify the recorder. Count aloud each vehicle in the queue as its front axle crosses the stop line and note the time of crossing. Call out the time of the fourth, 10th, and last vehicle in the stopped queue as its front axle crosses the stop line.

If queued vehicles are still entering the intersection at the end of the green interval, call out "saturation through the end of green—last vehicle was number XX." Note any unusual events that may have influenced the saturation flow rate, such as buses, stalled vehicles, and unloading trucks.

The period of saturation flow begins when the front axle of the fourth vehicle in the queue crosses the roadway reference line (e.g., stop line) and ends when the front axle of the last queued vehicle crosses this line. The last queued vehicle may be a vehicle that joined the queue during the green indication.

Data Reduction

Measurements are taken cycle by cycle. To reduce the data for each cycle, the time recorded for the fourth vehicle is subtracted from the time recorded for the last vehicle in the queue. This value represents the sum of the headways for the fifth through nth vehicle, where n is the number of the last vehicle surveyed (which may not be the last vehicle in the queue). This sum is divided by the number of headways after the fourth vehicle [i.e., divided by ($n - 4$)] to obtain the average headway per vehicle under saturation flow. The saturation flow rate is 3,600 divided by this average headway.

For example, if the time for the fourth vehicle was observed as 10.2 s and the time for the 14th and last vehicle surveyed was 36.5 s, the average saturation headway per vehicle is as follows:

$$\frac{(36.5 - 10.2)}{(14 - 4)} = \frac{26.3}{10} = 2.63 \text{ s/veh}$$

The prevailing saturation flow rate in that cycle is as follows:

$$\frac{3{,}600}{2.63} = 1{,}369 \text{ veh/h/ln}$$

To obtain a statistically significant value, a minimum of 15 signal cycles (each with more than eight vehicles in the initial queue) is typically required. The average of the saturation headway per vehicle values from the individual cycles is divided into 3,600 to obtain the prevailing saturation flow rate for the surveyed lane. The percentage of heavy vehicles and turning vehicles in the sample should be determined and noted for reference.

Calibration Technique

This subsection describes a technique for quantifying the base saturation flow rate at a local level. It consists of three tasks. The first task entails measuring the prevailing saturation flow rate at representative locations in the local area. The second task requires the calculation of an adjusted saturation flow rate for the same locations where a prevailing saturation flow rate was measured. The third task combines the information to compute the local base saturation flow rate.

This technique will require some resource investment by the agency. However, it should need to be completed only once every few years. In fact, it should be repeated only when there is evidence of a change in local driver behavior. The benefit of this calibration activity will be realized by the agency in terms of more accurate estimates of motorized vehicle performance, which should translate into more effective decisions related to infrastructure investment and system management.

Task 1. Measure Prevailing Saturation Flow Rate

This task requires measuring the prevailing saturation flow rate of one or more lane groups at each of several representative intersections in the local area. The minimum number of lane groups needed in the data set is difficult to judge for all situations; however, it should reflect a statistically valid sample. The data set should also provide a reasonable geographic and physical representation of the population of signalized intersections in the local area.

The lane groups for which the prevailing saturation flow rate is measured should include a representative mix of left-turn, through, and right-turn lane groups. It should not include left-turn lane groups that operate in the permitted or the protected-permitted mode or right-turn lane groups that have protected-permitted operation. These lane groups are excluded because of the complex nature of permitted and protected-permitted operation. The saturation flow rate for these lane groups tends to have a large amount of random variation that makes it more difficult to quantify the local base saturation flow rate with an acceptable level of precision.

Once the set of lane groups is identified, the technique described in the previous subsection is used to measure the prevailing saturation flow rate at each location.

Task 2. Compute Adjusted Saturation Flow Rate

For this task, the saturation flow rate calculation procedure in Chapter 19 is used to compute the adjusted saturation flow rate for each lane group in the data set. If a lane group is at an intersection with actuated control for one or more phases, the motorized vehicle methodology (as opposed to just the saturation flow rate procedure) will be needed to compute the adjusted saturation flow rate accurately. Regardless, the base saturation flow rate used with the procedure (or methodology) for this task must be 1,900 pc/h/ln.

Task 3. Compute Local Base Saturation Flow Rate

The local base saturation flow rate is computed with Equation 31-185.

Equation 31-185

$$s_{o,\text{local}} = 1{,}900 \frac{\sum_{i=1}^{m} s_{\text{prevailing},i}}{\sum_{i=1}^{m} s_i}$$

where

$s_{o,\text{local}}$ = local base saturation flow rate (pc/h/ln),

$s_{\text{prevailing},i}$ = prevailing saturation flow rate for lane group i (veh/h/ln),

s_i = (adjusted) saturation flow rate for lane group i (veh/h/ln), and

m = number of lane groups.

Once the local base saturation flow rate $s_{o,\text{local}}$ is quantified by this technique, it is substituted thereafter for s_o in any equation in an HCM chapter that refers to this variable.

7. COMPUTATIONAL ENGINE DOCUMENTATION

This section uses a series of flowcharts and linkage lists to document the logic flow for the computational engine.

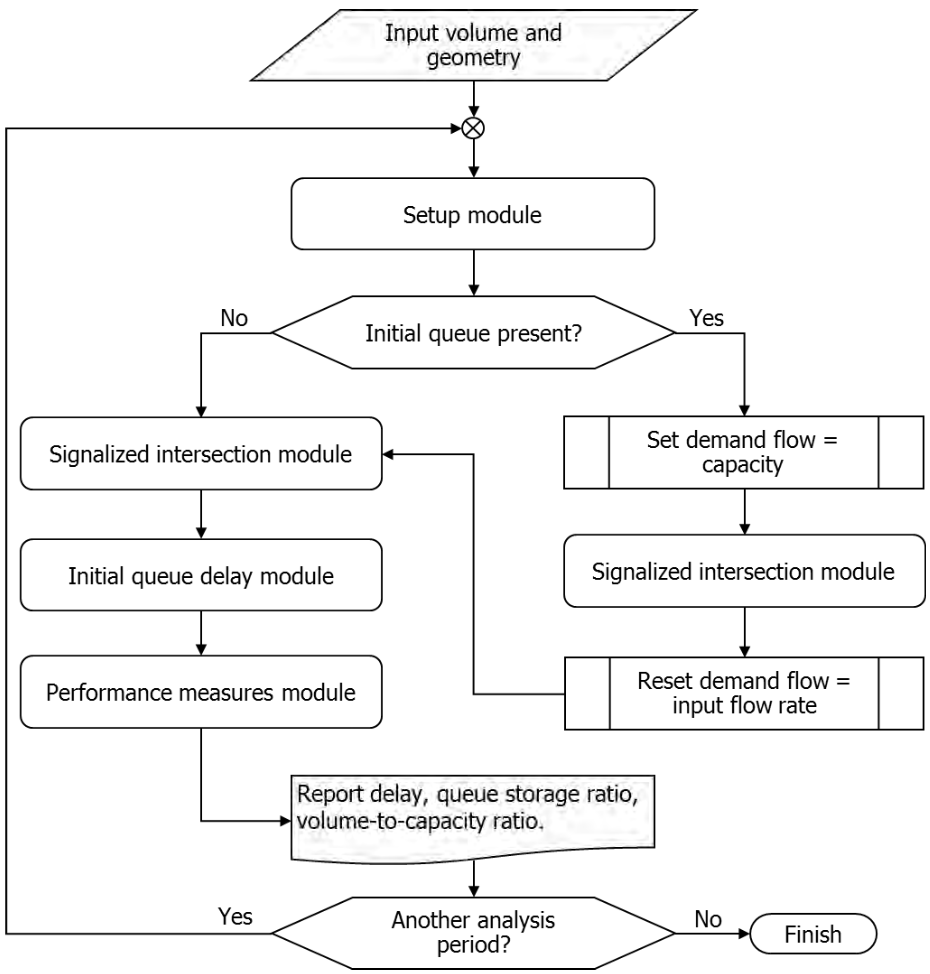
FLOWCHARTS

The methodology flowchart is shown in Exhibit 31-48. The methodology is shown to consist of four main modules:

- Setup module,
- Signalized intersection module,
- Initial queue delay module, and
- Performance measures module.

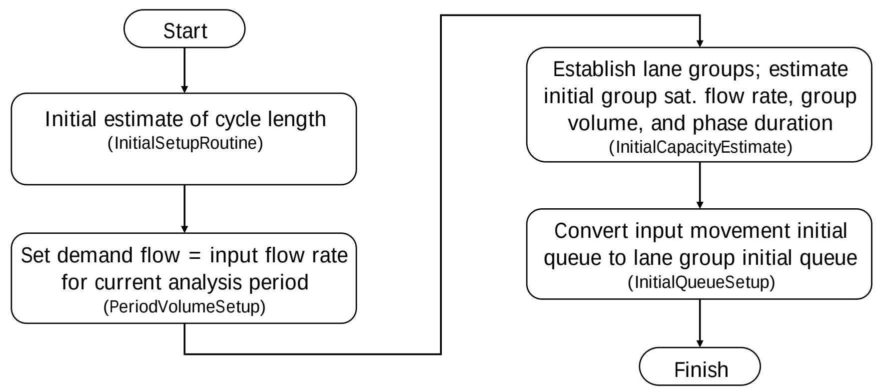
This subsection provides a separate flowchart for each of these modules.

Exhibit 31-48
Methodology Flowchart



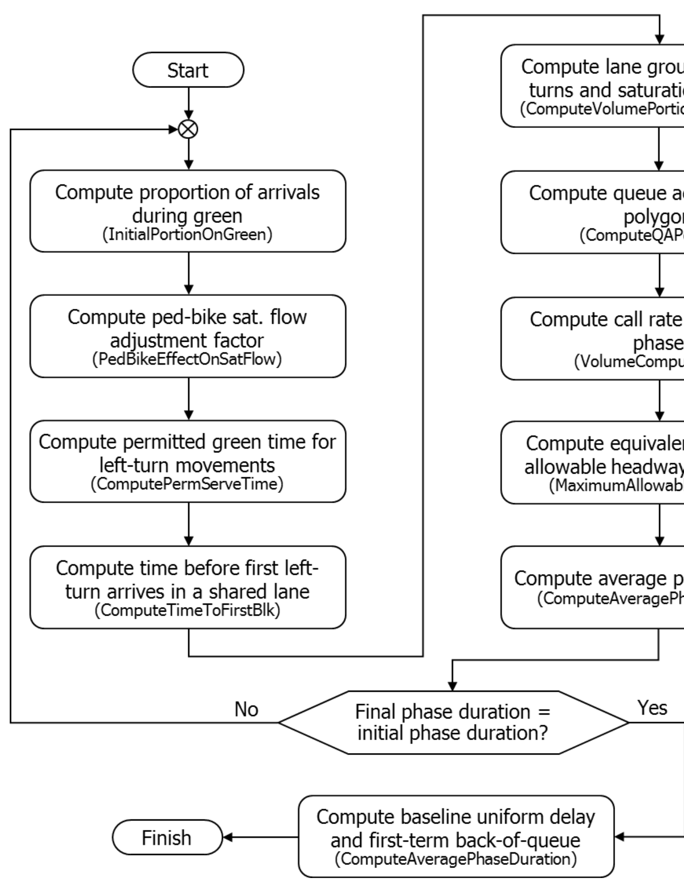
The setup module is shown in Exhibit 31-49. It consists of four main routines, as shown in the large rectangles of the exhibit. The main function of each routine, as well as the name given to it in the computational engine, is shown in the exhibit. These routines are described further in the next subsection.

Exhibit 31-49
Setup Module



The signalized intersection module is shown in Exhibit 31-50. It consists of nine main routines followed by a tenth and final computation routine performed after the final phase duration equals the initial phase duration. The main function of each routine, as well as the name given to it in the computational engine, is shown in the exhibit. These routines are described further in the next subsection.

Exhibit 31-50
Signalized Intersection Module



The initial queue delay module is shown in Exhibit 31-51. It consists of four main routines. The main function of each routine is shown in the exhibit.

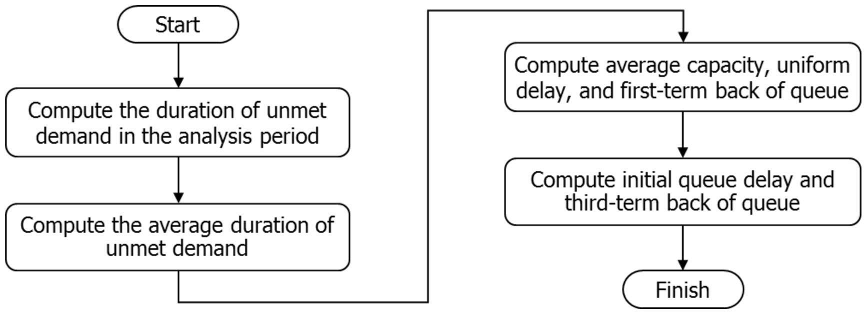


Exhibit 31-51
Initial Queue Delay Module

The performance measures module is shown in Exhibit 31-52. It consists of four main routines. The main function of each routine is shown in the exhibit. Two of the routines are complicated enough to justify their development as separate entities in the computational engine. The name given to each of these two routines is also shown in the exhibit, and they are described further in the next subsection.

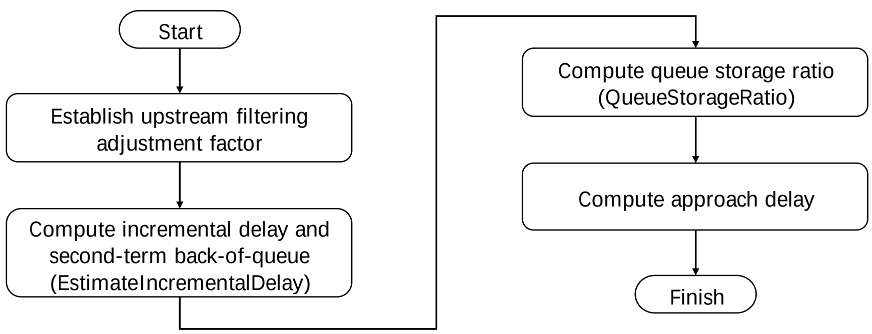


Exhibit 31-52
Performance Measures Module

LINKAGE LISTS

This subsection uses linkage lists to describe the main routines that compose the computational engine. Each list is provided in a table (an exhibit) that identifies the routine and the various subroutines to which it refers. Conditions for which the subroutines are used are also provided.

The lists are organized by module, as described in the previous subsection. Four tables are provided to address the following three modules:

- Setup module (one table),
- Signalized intersection module (two tables), and
- Performance measures module (one table).

The initial queue delay module does not have a linkage list because it does not call any specific routines.

The linkage list for the setup module is provided in Exhibit 31-53. The main routines are listed in the far-left column of the exhibit and are identified in Exhibit 31-49.

Exhibit 31-53
Setup Module Routines

Routine	Subroutine	Conditions for Use
InitialSetupRoutine	Compute change period ($Y + R_c$).	None
	Compute initial estimate of cycle length C.	None
PeriodVolumeSetup	a. Compute period volume before initial queue analysis, and b. Restore period volume if initial queue analysis conducted.	Used for multiple-period analysis
	a. Save input volume as it will be overwritten if initial queue is present, and b. Restore input volume if initial queue analysis conducted.	Used for single-period analysis
InitialCapacityEstimate	getPermissiveLeftServiceTime (computes g_u, the duration of the permitted period that is not blocked by an opposing queue)	Used if subject phase serves a left-turn movement with (*a*) permitted mode or (*b*) protected-permitted mode
	getPermissiveLeftEffGreen (computes g_p, the duration of the permitted green for permitted left-turn movements)	Used if subject phase serves a left-turn movement with (*a*) permitted mode or (*b*) protected-permitted mode
	Define lane groups for each approach.	None
	Establish initial estimate of lane group volume, saturation flow rate, and number of lanes capacity.	None
	Establish initial estimate of proportion of turns in a shared-lane lane group.	Used for shared-lane lane groups
	PermittedSatFlow (computes permitted left-turn saturation flow rate s_p)	Used if lane group serves a left-turn movement with protected-permitted mode
	getParkBusSatFlowAdj (computes combined parking and bus blockage saturation flow adjustment factors)	Used if lane group is adjacent to on-street parking or a local bus stop
	Establish initial estimate of queue service time g_s.	None
InitialQueueSetup	Distribute input movement initial queue to corresponding lane groups.	Used for first analysis period
	Assign residual queue from last period to initial queue of current period, and distribute initial queue among affected lane groups.	Used for second and subsequent analysis periods

The linkage list for the signalized intersection module is provided in Exhibit 31-54. The main routines are listed in the far-left column of the exhibit and are identified in Exhibit 31-50. The ComputeQAPolygon routine is complex enough to justify the presentation of its subroutines in a separate linkage list. This supplemental list is provided in Exhibit 31-55.

Exhibit 31-54
Signalized Intersection Module: Main Routines

Routine	Subroutine	Conditions for Use
InitialPortionOnGreen	Compute portion arriving during green *P*.	None
PedBikeEffectOnSatFlow	PedBikeEffectOnLefts	Used if subject phase serves a left-turn movement with (*a*) permitted mode or (*b*) protected-permitted mode
	PedBikeEffectOnRights	Used if subject phase serves a right-turn movement
	PedBikeEffectOnLeftsUnopposed	Used if subject phase serves a left-turn movement with split phasing
ComputePermServeTime	getPermissiveLeftServiceTime (computes g_u, the duration of the permitted period that is not blocked by an opposing queue)	Used if subject phase serves a left-turn movement with (*a*) permitted mode or (*b*) protected-permitted mode
	getPermissiveLeftEffGreen (computes g_p, the duration of the permitted green for permitted left-turn movements)	Used if subject phase serves a left-turn movement with (*a*) permitted mode or (*b*) protected-permitted mode
ComputeTimeToFirstBlk	getTimetoFirstBlk (computes g_f, the time before the first left-turning vehicle arrives and blocks the shared lane)	Used if subject phase serves a left-turn movement in a shared lane with (*a*) permitted mode or (*b*) protected-permitted mode
ComputeVolumePortions-AndSatFlow	PermittedSatFlow (computes permitted left-turn saturation flow rate s_p)	Used if lane group serves a left-turn movement with protected-permitted mode
	PortionTurnsInSharedTRlane (computes proportion of right-turning vehicles in shared lane P_R)	Used if approach has exclusive left-turn lane and subject lane group is a shared lane serving through and right-turning vehicles
	SatFlowforPermExclLefts	Used if lane group serves a left-turn movement with a permitted mode in an exclusive lane
	PortionTurnsInSharedLTRlane (computes proportion of right-turning vehicles in shared lane P_R and proportion of left-turning vehicles in shared lane P_L)	Used if approach has a shared lane serving left-turn and through vehicles
ComputeQAPolygon	QAP_ProtPermExclLane	Used if lane group serves a left-turn movement in an exclusive lane with the protected-permitted mode
	QAP_ProtMvmtExclLane	Used if lane group's movement has an exclusive lane and is served with protected mode
	QAP_ProtSharedLane	Used if lane group has (*a*) a shared lane with through and right-turning movements or (*b*) a shared lane with through and left-turning movements served with split phasing
	QAP_PermLeftExclLane	Used if lane group serves a left-turn movement in an exclusive lane with the permitted mode
	QAP_PermSharedLane	Used if lane group serves a left-turn movement in a shared lane with the permitted mode

Exhibit 31-54 (continued)
Signalized Intersection
Module: Main Routines

Routine	Subroutine	Conditions for Use
VolumeComputations	Determine call rate to extend green λ.	None
	Determine call rate to activate a phase q_v, q_p.	None
MaximumAllowable-Headway	Compute maximum allowable headway for each lane group *MAH*.	Calculations vary depending on lane group movements, lane assignment, phase sequence, and left-turn operational mode.
	Compute equivalent maximum allowable headway for each phase and timer *MAH**.	None
ComputeAverage-PhaseDuration	Compute probability of green extension p.	Computed for all phases except for the timer that serves the protected left-turn movement in a shared lane
	Compute maximum queue service time for all lane groups served during the phase.	None
	Compute probability of phase termination by extension to maximum limit (i.e., max-out).	None
	Compute green extension time g_e.	None
	Compute probability of a phase call p_c.	None
	Compute unbalanced green duration G_u.	None
	Compute average phase duration D_p.	None

Exhibit 31-55
Signalized Intersection Module: ComputeQAPolygon Routines

Routine	Subroutine	Conditions for Use
QAP_ProtPermExclLane	ADP_ProtPermExcl (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with left-turn movements in exclusive lane and served by protected-permitted mode
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_ProtMvmtExclLane	ADP_ProtMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with one service period
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_ProtSharedLane	ADP_ProtMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with one service period
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_PermLeftExclLane	ADP_PermLeftExclLane (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with left-turn movements in exclusive lane and served by permitted mode
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None
QAP_PermSharedLane	ADP_PermSharedMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for shared-lane lane groups with a permitted left-turn movement
	ADP_ProtMvmt (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with one service period
	ADP_ProtPermShared (compute baseline first-term back-of-queue estimate Q_{1b})	Used for lane groups with left-turn movements in shared-lane lane group and served by protected-permitted mode
	getUniformDelay (compute baseline uniform delay d_{1b})	None
	Compute queue service time g_s.	None
	Compute lane group available capacity.	None
	Compute movement capacity.	None

The linkage list for the performance measures module is provided in Exhibit 31-56. The main routines are listed in the far-left column and are identified in Exhibit 31-52.

Exhibit 31-56
Performance Measures Module Routines

Routine	Subroutine	Conditions for Use
EstimateIncrementalDelay	Compute incremental delay d_2 and second-term back-of-queue estimate Q_2.	None
QueueStorageRatio	Compute queue storage ratio L_Q.	None

8. USE OF ALTERNATIVE TOOLS

This section illustrates the use of alternative evaluation tools to evaluate the operation of a signalized intersection. The intersection described in Example Problem 1 of Section 9 is used for this purpose. There are no limitations in this example that would suggest the need for alternative tools. However, it is possible to introduce situations, such as short left-turn bays, for which an alternative tool might provide a more realistic assessment of intersection operation.

The basic layout of the example intersection is shown in the second exhibit of Example Problem 1 of Section 9. The left-turn movements on the north–south street operate under protected-permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the east–west street operate as permitted. To simplify the discussion, the pedestrian and parking activity is removed. A pretimed signal operation is used.

EFFECT OF STORAGE BAY OVERFLOW

The effect of left-turn storage bay overflow is described in this subsection as a means of illustrating the use of alternative tools. The motorized vehicle methodology in Chapter 19 can be used to compute a queue storage ratio that compares the back-of-queue estimate with the available storage length. This ratio is used to identify bays that have inadequate storage. Overflow from a storage bay can be expected to reduce approach capacity and increase the approach delay. However, these effects of bay overflow are not addressed by the motorized vehicle methodology.

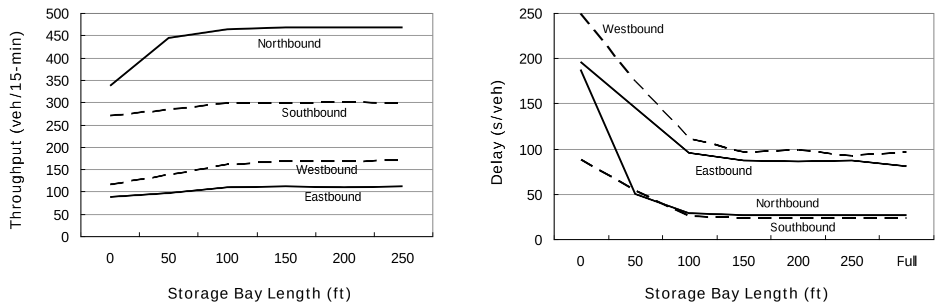
Effect of Overflow on Approach Throughput and Delay

A simulation software product was selected as the alternative tool for this analysis. The intersection was simulated for a range of storage bay lengths from 0 to 250 ft. All other input data remained the same. The results presented here represent the average of 30 simulation runs for each case.

The effect of bay overflow was assessed by examining the relationship between bay length, approach throughput, and approach delay. Exhibit 31-57 shows this effect. The throughput on each approach is equal to the demand volume when storage is adequate but drops off when the bay length is decreased.

A delay comparison is also presented in Exhibit 31-57. The delay on each approach increases as bay length is reduced. The highest delay is associated with a zero-length bay, which is effectively a shared lane. The zero-length case is included here to establish a boundary condition. The delay value becomes excessive when overflow occurs. This situation often degrades into oversaturation, and a proper assessment of delay would require a multiple-period analysis to account for the buildup of long-term queues.

Exhibit 31-57
Effect of Storage Bay Length on Throughput and Delay



(a) Throughput (b) Delay

For case-specific applications, parameters that could influence the evaluation of bay overflow include the following:

- Number of lanes for each movement,
- Demand volumes for each movement,
- Impedance of left-turning vehicles by oncoming traffic during permitted periods,
- Signal-timing plan (cycle length and phase times),
- Factors that affect the number of left-turn sneakers for left-turn movements that have permitted operation, and
- Other factors that influence the saturation flow rates.

The example intersection described here had two through lanes in all directions. If only one through lane had existed, the blockage effect would have been much more severe.

Effect of Overflow on Through Movement Capacity

This subsection illustrates how an alternative tool can be used to model congestion due to storage bay overflow. An example was set up involving constant blockage of a through lane by left-turning vehicles. This condition arises only under very severe oversaturation.

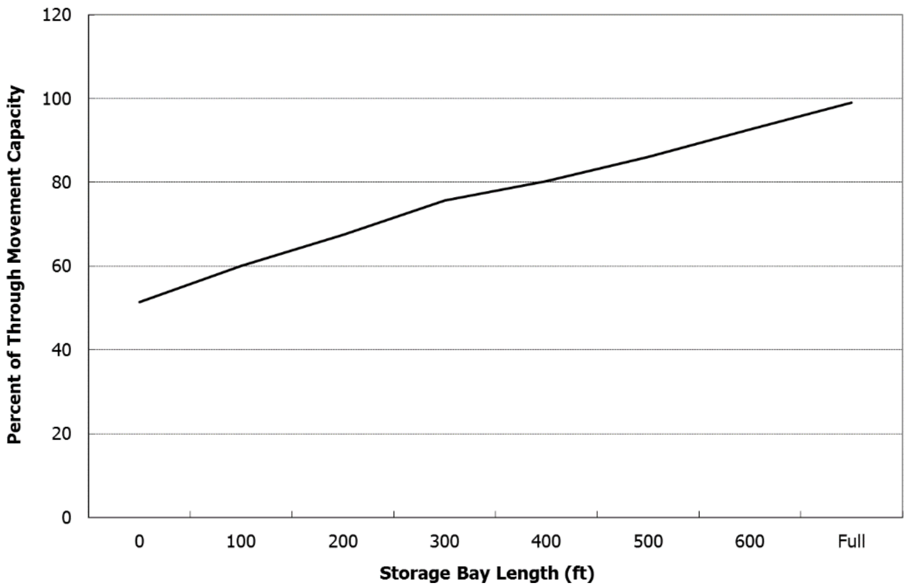
The following variables are used for this examination:

- Cycle length is 90 s,
- Effective green time is 41 s, and
- Saturation flow rate is 1,800 veh/h/ln.

The approach has two through lanes. Traffic volumes were sufficient to overload both lanes, so that the number of trips processed by the simulation model was determined to be an indication of through movement capacity. With no storage bay overflow effect, this capacity is computed as 1,640 veh/h (= 3,600 × 41/90). So, in a 15-min period, 410 trips were processed on average when there was no overflow.

Exhibit 31-58 shows the effect of the storage bay length on the through movement capacity. The percentage of the full capacity is plotted as a function of the storage bay length over the range of 0 to 600 ft. As expected, a zero-length bay reduces the capacity to 50% of its full value because one lane would be constantly blocked. At the other extreme, the "no blockage" condition, achieved by setting the left-turn volume to zero, indicates the full capacity was available. The loss of capacity is more or less linear for storage lengths up to 600 ft, at which point about 90% of the full capacity is achieved.

Exhibit 31-58
Effect of Storage Bay Length on Capacity



Bay overflow is a very difficult phenomenon to deal with analytically, and a substantial variation in its treatment is expected among alternative tools. The main issue for modeling is the behavior of left-turning drivers denied access to the left-turn bay because of the overflow. The animated graphics display produced by some tools can often be used to examine this behavior and assess the tool's validity. Typically, some model parameters can be adjusted so that the resulting behavior is more realistic.

EFFECT OF RIGHT-TURN-ON-RED OPERATION

The treatment of right-turn-on-red (RTOR) operation in the motorized vehicle methodology is limited to the removal of RTOR vehicles from the right-turn demand volume. If the right-turn movement is served by an exclusive lane, the methodology suggests RTOR volume can be estimated as equal to the left-turn demand of the complementary cross street left-turn movement, whenever this movement is provided a left-turn phase. Given the simplicity of this treatment, it may be preferable to use an alternative tool to evaluate RTOR operation under the following conditions:

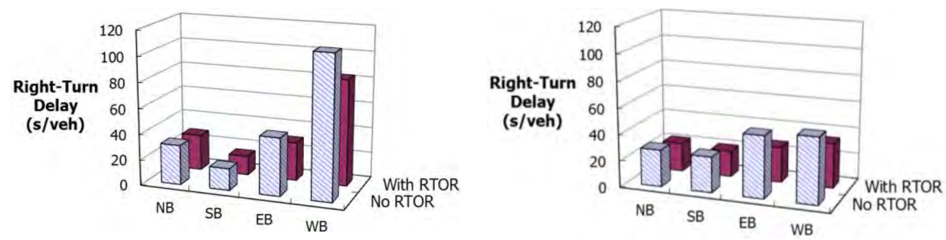
- RTOR operation occurs at the intersection,
- Right turns are a critical element of the operation,
- An acceptable LOS depends on RTOR movements, or
- Detailed phasing alternatives involving RTOR are being considered.

The remainder of this subsection examines the RTOR treatment offered in the motorized vehicle methodology. The objective of this discussion is to illustrate when alternative tools should be considered.

Effect of Right-Turn Lane Allocation

This subsection examines the effect of the lane allocation for the right-turn movement. The lane-allocation scenarios considered include (*a*) provision of a shared lane for the right-turn movement and (*b*) provision of an exclusive right-turn lane. Exhibit 31-59 shows the results of the analysis. The intersection was simulated with (and without) the RTOR volume.

Exhibit 31-59
Effect of Right-Turn-on-Red and Lane Allocation on Delay



(a) Shared Lane (b) Exclusive Right-Turn Lane

The trends in Exhibit 31-59 indicate there are only minimal differences in delay when RTOR is allowed relative to when it is not allowed. The northbound and southbound approaches had no shadowing opportunities because the eastbound and westbound movements did not have a protected left-turn phase. As a result, the effect of lane allocation and RTOR operation was negligible for the northbound and southbound right-turn movements.

In contrast, the eastbound and westbound right-turn movements were shadowed by the protected left-turn phases for the northbound and southbound approaches. As a result, the effect of lane allocation was more notable for the eastbound and the westbound right-turn movements.

Effect of Right-Turn Demand Volume

This subsection examines the effect of right-turn demand volume on right-turn delay, with and without RTOR allowed. The right-turn volumes varied from 100 to 400 veh/h on all approaches. Exclusive right-turn storage bays were provided on each approach.

The results are shown in Exhibit 31-60. They indicate delay to the northbound and southbound right-turn movements was fairly insensitive to right-turn volume, with or without RTOR allowed. The available green time on these approaches provided adequate capacity for the right turns. RTOR operation provided about a 25% delay reduction.

The delay to the eastbound and westbound right-turn movements increased rapidly with right-turn volume when RTOR was not allowed. At 300 veh/h and no RTOR, the right-turn delay becomes excessive in both directions. With RTOR, delay is less sensitive to right-turn volume. This trend indicates the additional capacity provided by RTOR is beneficial for higher right-turn volume levels.

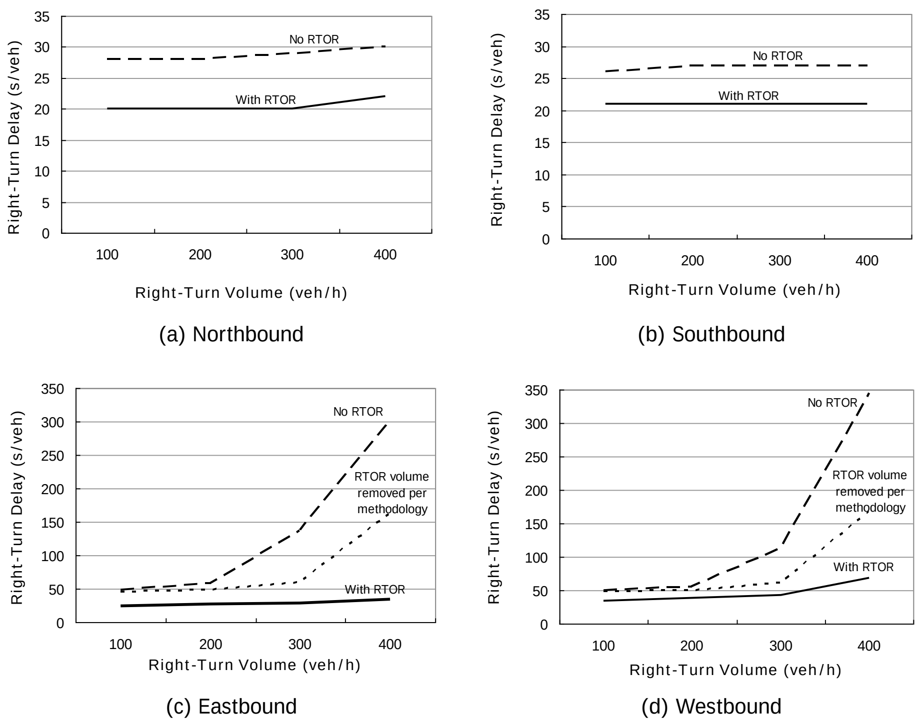


Exhibit 31-60
Effect of Right-Turn-on-Red and Right-Turn Volume on Delay

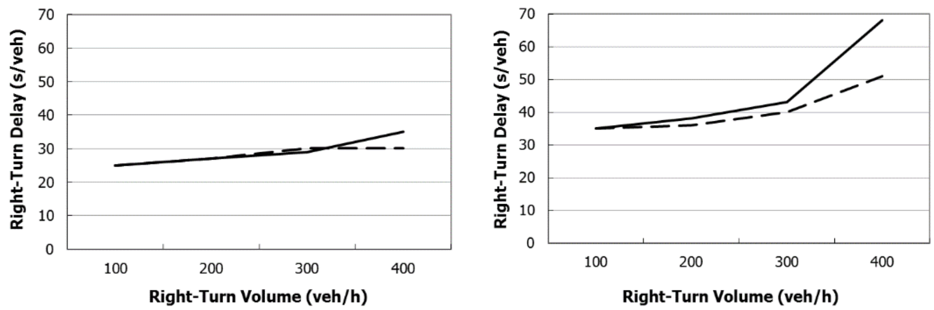
The treatment of RTOR suggested in the motorized vehicle methodology (i.e., removal of the RTOR vehicles from the right-turn volume) was also examined. The simulation analysis was repeated with the right-turn volumes reduced in this manner to explore the validity of this treatment.

The results of this analysis are shown in Exhibit 31-60 for the eastbound and westbound approaches. The trends shown suggest the treatment yields a result that is closer to the "with RTOR" case, as intended. However, use of the treatment in this case could still lead to erroneous conclusions about right-turn delay at intersections with high right-turn volumes.

Effect of a Protected Right-Turn Phase

This subsection compares the effect of adding a protected right-turn phase without RTOR allowed relative to just allowing RTOR. The example intersection was modified to include an exclusive right-turn storage bay and a protected right-turn phase for both the eastbound and westbound approaches. Each phase was timed concurrently with the complementary northbound or southbound left-turn phase, as appropriate. The results are shown in Exhibit 31-61. The trends in the exhibit indicate the protected phase does not improve over RTOR operation at low volume levels. However, it does provide some delay reduction at the high end of the volume scale.

Exhibit 31-61
Effect of Right-Turn-on-Red and Right-Turn Protection on Delay



(a) Eastbound (b) Westbound

This examination indicates RTOR operation can have some effect on right-turn delay. The effect is most notable when there are no shadowing Δopportunities in the phase sequence for right-turn service or the right-turn volume is high. The use of an alternative tool to evaluate RTOR operation may provide a more realistic estimate of delay than simply removing RTOR vehicles from the right-turn demand volume, as suggested in Chapter 19.

EFFECT OF SHORT THROUGH LANES

One identified limitation of the motorized vehicle methodology is its inability to evaluate short through lanes that are added or dropped at the intersection. This subsection describes the results from an evaluation of this geometry for the purpose of illustrating the effect of short through lanes.

Several alternative tools can address the effect of short through lanes. Each tool will have its own unique method of representing lane drop or add geometry and models of driver behavior. Some degree of approximation is involved with all evaluation tools.

The question under consideration is, "How much additional through traffic could the northbound approach accommodate if a lane were added both 150 ft upstream and 150 ft downstream of the intersection?" The capacity of the original two northbound lanes was computed as 1,778 veh/h (i.e., 889 veh/h/ln) by using the motorized vehicle methodology. The simulation tool's start-up lost time and saturation headway parameters were then adjusted so the simulation tool produced the same capacity. It was found in this case that a 2.3-s headway and 3.9-s start-up lost time produced the desired capacity.

Finally, the additional through lane was added to the simulated intersection, and the process of determining capacity was repeated. On the basis of an average of 30 runs, the capacity of the additional lane was computed as 310 veh/h. Theoretically, the addition of a full lane would increase the capacity by another 889 veh/h, for a total of 2,667 veh/h.

The alternative tool indicates the additional lane contributes only 0.35 equivalent lane (= 310/889). This result cannot be stated as a general conclusion that applies to all cases because other parameters (such as the signal-timing plan and the proportion of right turns in the lane group) will influence the results. More important, the results are likely to vary among alternative tools given the likely differences in their driver behavior models.

EFFECT OF CLOSELY SPACED INTERSECTIONS

The effect of closely spaced intersections is examined in this subsection. The motorized vehicle methodology does not account for the effect of queue cyclic spillback from a downstream signal or demand starvation from an upstream signal. It is generally accepted that simulation of these effects is desirable when two closely spaced signalized intersections interact with each other in this manner.

Consider two intersections separated by 200 ft along the north–south roadway. They operate with the same cycle length and the same northbound and southbound green time. To keep the problem simple, only through movements are allowed at these intersections. The northbound approach is used in this discussion to illustrate the effect of the adjacent intersection. The layout of this system and the resulting lane blockage are illustrated in Exhibit 31-62.

Exhibit 31-62
Closely Spaced Intersections

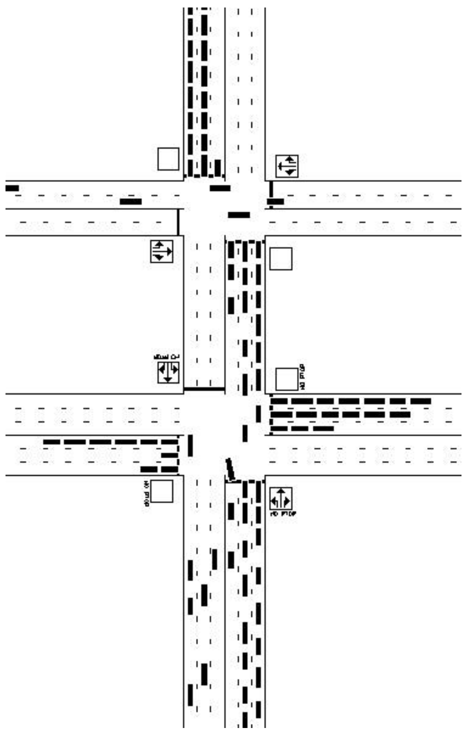
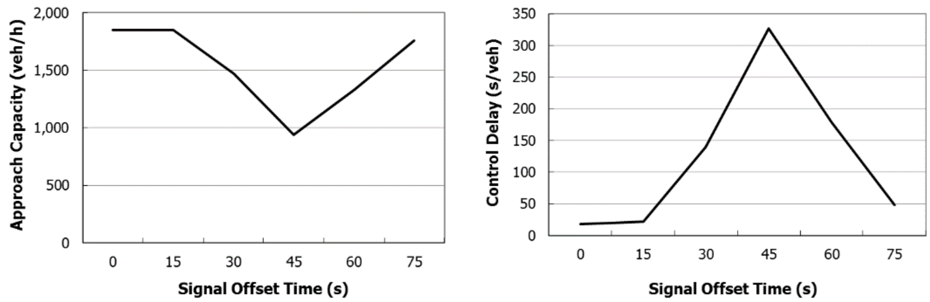


Exhibit 31-62 illustrates both cyclic spillback and demand starvation at one point in the cycle. For the northbound direction, traffic queues have spilled back from the downstream intersection to block the upstream intersection. For the southbound direction, the traffic at the upstream intersection is prevented from reaching the downstream intersection by the red signal at the upstream intersection. Valuable green time is being wasted in both travel directions at the southern intersection.

Exhibit 31-63 illustrates the relationship between signal offset and the performance of the northbound travel direction. In terms of capacity, the exhibit shows that under the best-case condition (i.e., zero offset), the capacity is maintained at a value slightly above the demand volume. Under the worst-case condition, the capacity is reduced to slightly below 1,000 veh/h. The demand volume-to-capacity ratio under this condition is about 1.7.

Exhibit 31-63
Effect of Closely Spaced Intersections on Capacity and Delay



(a) Approach Capacity (b) Control Delay

The effect of signal offset time on the delay to northbound traffic approaching the first intersection is also shown in Exhibit 31-63. As expected, the delay is minimal under favorable offsets, but it increases rapidly as the offset becomes less favorable. Delay is at its maximum value with a 45-s offset time. The large value of delay suggests that approach is severely oversaturated.

The delay reported by most simulation tools represents the delay incurred by vehicles when they *depart* the system during the analysis period, as opposed to the delay incurred by vehicles that *arrive* during the analysis period. The latter measure represents the delay reported by the motorized vehicle methodology.

For oversaturated conditions, the delay reported by a simulation tool may be biased when the street system is not adequately represented. This bias occurs when the street system represented to the tool does not physically extend beyond the limits of the longest queue that occurs during the analysis period.

The issues highlighted in the preceding paragraphs must be considered when an alternative tool is used. Specifically, a multiple-period analysis must be conducted that temporally spans the period of oversaturation. Also, the spatial boundaries of the street system must be large enough to encompass all queues during the saturated time periods. A more detailed discussion of multiple-period analyses is presented in Chapter 7, Interpreting HCM and Alternative Tool Results.

9. EXAMPLE PROBLEMS

This section describes the application of each of the motorized vehicle, pedestrian, and bicycle methodologies through the use of example problems. Exhibit 31-64 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operational analysis level in terms of the calculations, except that default values are used when field-measured values are not available.

Exhibit 31-64
Example Problems

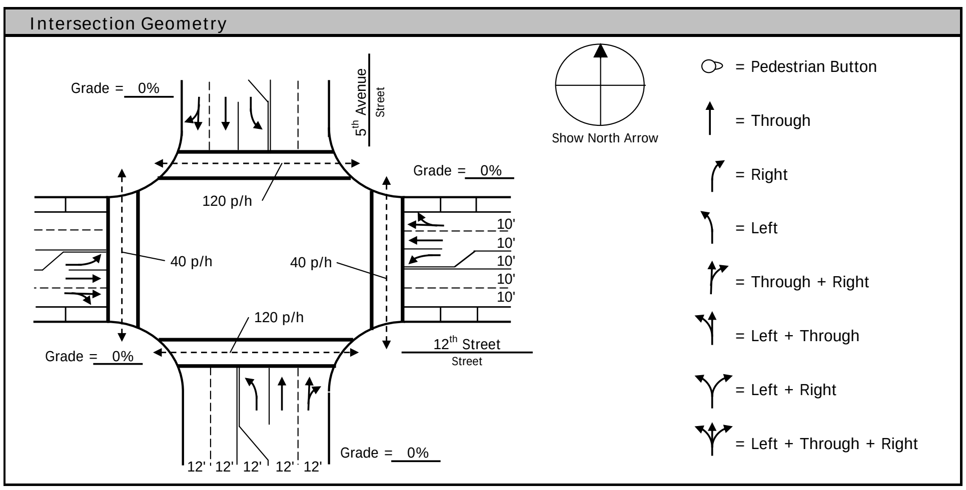
Problem Number	Description	Analysis Level
1	Motorized vehicle LOS	Operational
2	Pedestrian delay, LOS, and circulation area	Operational
3	Bicycle LOS	Operational
4	Pedestrian delay, two-stage crossing of one intersection leg	Operational
5	Pedestrian delay, two-stage crossing of two intersection legs	Operational

EXAMPLE PROBLEM 1: MOTORIZED VEHICLE LOS

The Intersection

The intersection of 5th Avenue and 12th Street is an intersection of two urban arterial streets. The intersection plan view is shown in Exhibit 31-65.

Exhibit 31-65
Example Problem 1:
Intersection Plan View



The Question

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

The Facts

The intersection's traffic, geometric, and signalization conditions are listed in Exhibit 31-66, Exhibit 31-67, and Exhibit 31-68, respectively. Exhibit 31-69 presents additional data. The volume data provided represent the demand flow rate during the 0.25-h analysis period, so a peak hour factor is not applicable to this evaluation.

Exhibit 31-66
Example Problem 1: Traffic Characteristics Data

Input Data Element	Eastbound L	Eastbound T	Eastbound R	Westbound L	Westbound T	Westbound R	Northbound L	Northbound T	Northbound R	Southbound L	Southbound T	Southbound R
Demand flow rate (veh/h)	71	318	106	118	600	24	133	1644	111	194	933	111
RTOR flow rate (veh/h)			0			0			22			33
Percentage heavy vehicles (%)	5	5		5	5		2	2		2	2	
Platoon ratio	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Upstream filtering adjustment factor	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Initial queue (veh)	0	0		0	0		0	0		0	0	
Base saturation flow rate (pc/h/ln)	1,900	1,900		1,900	1,900		1,900	1,900		1,900	1,900	
Pedestrian flow rate (p/h)		120			120			40			40	
Bicycle flow rate (bicycles/h)		0			0			0			0	
On-street parking maneuver rate (maneuvers/h)		5			5							
Local bus stopping rate (buses/h)		0			0			0			0	

Note: L = left turn; T = through; R = right turn.

Exhibit 31-67
Example Problem 1: Geometric Design Data

Input Data Element	Eastbound L	Eastbound T	Eastbound R	Westbound L	Westbound T	Westbound R	Northbound L	Northbound T	Northbound R	Southbound L	Southbound T	Southbound R
Number of lanes (ln)	1	2	0	1	2	0	1	2	0	1	2	0
Average lane width (ft)	10.0	10.0		10.0	10.0		12.0	12.0		12.0	12.0	
Number of receiving lanes (ln)		2			2			2			2	
Turn bay length (ft)	200			200			200			200		
Presence of on-street parking	No		Yes	No		Yes	No		No	No		No
Approach grade (%)		0			0			0			0	

Note: L = left turn; T = through; R = right turn.

Exhibit 31-68
Example Problem 1: Signal Control Data

Input Data Element	Eastbound		Westbound		Northbound		Southbound	
Type of signal control	Actuated		Actuated		Actuated		Actuated	
Phase sequence	No left-turn phase		No left-turn phase		Leading left		Lagging left	
Phase number		2		6	3	8	7	4
Movement		L+T+R		L+T+R	L	T+R	L	T+R
Left-turn operational mode		Perm.		Perm.	Prot.-Perm.		Prot.-Perm.	
Dallas left-turn phasing option					No		No	
Passage time (s)		2.0		2.0	2.0	2.0	2.0	2.0
Maximum green (s)		30		30	25	50	25	50
Minimum green (s)		5		5	5	5	5	5
Yellow change (s)		4.0		4.0	4.0	4.0	4.0	4.0
Red clearance (s)		0		0	0	0	0	0
Walk (s)		5		5		5		5
Pedestrian clear (s)		14		14		16		16
Phase recall		No		No	No	No	No	No
Dual entry		Yes		Yes	No	Yes	No	Yes
Simultaneous gap-out	Yes				Yes			

Note: L = left turn; T = through; R = right turn; Prot. = protected; Perm. = permitted.

Exhibit 31-69
Example Problem 1: Other Data

Input Data Element	Eastbound L	Eastbound T	Eastbound R	Westbound L	Westbound T	Westbound R	Northbound L	Northbound T	Northbound R	Southbound L	Southbound T	Southbound R
Analysis period duration (h)		0.25			0.25			0.25			0.25	
Speed limit (mi/h)		35			35			35			35	
Stop-line detector length (ft)	40	40		40	40		40	40		40	40	
Detection mode	Pres.	Presence		Pres.	Presence		Pres.	Presence		Pres.	Presence	
Area type	Central business district											

Note: L = left turn; T = through; R = right turn; Pres. = presence.

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

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

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

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

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

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

As noted in the next section, none of the intersection movements have two or more exclusive lanes. For this reason, the saturation flow rate adjustment factor for lane utilization is not applicable. Any unequal lane use that may occur due to the shared through and right-turn lane groups will be accounted for in the lane group flow rate calculation, as described in the Lane Group Flow Rate on Multiple-Lane Approaches subsection of Section 2.

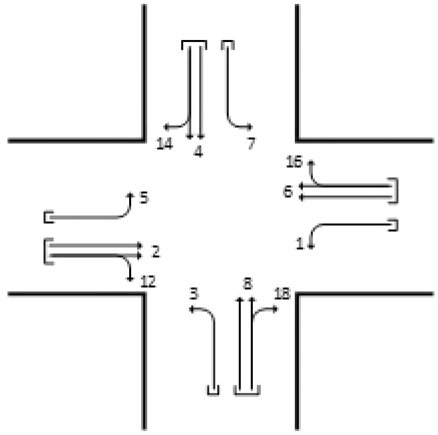
Outline of Solution

The solution follows the steps listed in Exhibit 19-18 of Chapter 19.

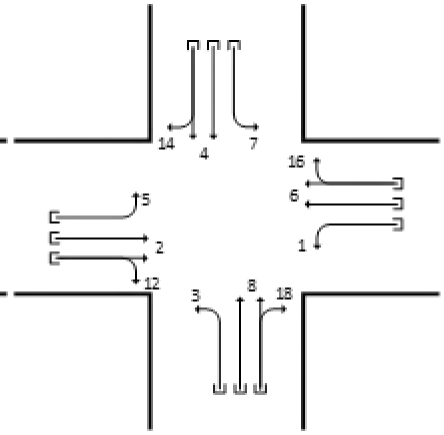
Step 1: Determine Movement Groups and Lane Groups

The left-turn lanes are designated as separate movement groups according to the rules described in Chapter 19. The through and shared right-turn and through lanes are combined into one movement group on each approach. The movement group designations are shown in Exhibit 31-70a with brackets showing how the individual movements are combined into movement groups.

Exhibit 31-70
Example Problem 1: Movement Groups and Lane Groups



(a) Movement Groups



(b) Lane Groups

Each lane is analyzed as a separate lane group according to the rules in Chapter 19. The lane group designations are shown in Exhibit 31-70b with brackets showing how the individual lanes are combined into lane groups.

Step 2: Determine Movement Group Flow Rate

Exhibit 31-71 shows the movement group flow rates, which are based on the movement groups identified in Exhibit 31-70a. The RTOR flow rate is subtracted from the right-turn volume for the northbound and southbound through-and-right-turn movement groups.

Exhibit 31-71
Example Problem 1: Movement Group Flow Rates

Data Element	Eastbound		Westbound		Northbound		Southbound	
Movement group	L	T+R	L	T+R	L	T+R	L	T+R
Number of lanes (ln)	1	2	1	2	1	2	1	2
Movement group flow rate (veh/h)	71	318 + 106 = 424	118	600 + 24 = 624	133	1,644 + 111 − 22 = 1,733	194	933 + 111 − 33 = 1,011

Note: L = left turn; T+R = combined through and right turn.

Step 3: Determine Lane Group Flow Rate

There is one shared lane and two or more lanes on each intersection approach. For this configuration, the lane group flow rates for the through-and-right-turn movement groups are computed by the procedures in the Lane Group Flow Rate on Multiple-Lane Approaches subsection of Section 2. The results of these calculations are given in Exhibit 31-72. The left-turn lane group volumes remain unchanged from Exhibit 31-71 because the movement groups and the lane groups are the same for the left-turn lanes. The volumes shown for the through lane group and the shared lane group represent the flow rates obtained from the Section 2 procedure.

Exhibit 31-72
Example Problem 1: Lane Group Flow Rates

Data Element	Eastbound			Westbound			Northbound			Southbound		
Lane group	L	T	T+R	L	T	T+R	L	T	T+R	L	T	T+R
Number of lanes (ln)	1	1	1	1	1	1	1	1	1	1	1	1
Flow rate (veh/h)	71	239	185	118	337	287	133	870	863	194	513	497

Note: L = left turn; T = through; T+R = combined through and right turn.

Step 4: Determine Adjusted Saturation Flow Fate

The base saturation flow rate is 1,900 veh/h/ln for each lane group. Adjustments made for each of the lane groups are summarized in the following paragraphs.

The left-turn lane groups for the eastbound and westbound approaches operate with the permitted mode. The saturation flow rate of a permitted left-turn movement s_p is determined with Equation 31-100. For example, the saturation flow rate for the eastbound left-turn lane group is computed with the following equation.

$$s_p = \frac{v_o e^{-v_o t_{cg}/3{,}600}}{1 - e^{-v_o t_{fh}/3{,}600}} = \frac{624 e^{-624(4.5)/3{,}600}}{1 - e^{-624(2.5)/3{,}600}} = 813 \text{ veh/h/ln}$$

The adjustment factor for the existence of parking and parking activity f_p is applied to the shared-lane lane groups for the eastbound and westbound approaches. This factor is computed with Equation 19-11.

The adjustment factor for area type f_a is applied to all lane groups. Guidance for determining this factor's value is provided in Section 3 of Chapter 19 (in the subsection titled Adjustment for Area Type).

The adjustment factor for heavy vehicles and grade f_{HVg} is computed with Equation 19-10. This factor is applicable to all lane groups.

The adjustment factors and the adjusted saturation flow rate for each movement are shown in Exhibit 31-73.

Exhibit 31-73
Example Problem 1: Adjusted Saturation Flow Rate

Data Element	Eastbound			Westbound			Northbound			Southbound		
Lane group	L	T	T+R	L	T	T+R	L	T	T+R	L	T	T+R
Phase number	2	2	2	6	6	6	3	8	8	7	4	4
Base saturation flow rate s_o (pc/h/ln)		1,900	1,900		1,900	1,900	1,900	1,900	1,900	1,900	1,900	1,900
Permitted left turn saturation flow rate s_p (veh/h/ln)	813			978								
Adjustment factor for left-turn vehicle presence, f_{LT}							0.95			0.95		
Adjustment factor for heavy vehicles and grade, f_{HVg}	0.96	0.96	0.96	0.96	0.96	0.96	0.98	0.98	0.98	0.98	0.98	0.98
Adjustment factor for existence of parking lane and parking activity, f_p			0.88			0.88						
Adjustment factor for area type, f_a	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Pedestrian adjustment factor for left-turn groups, f_{Lpb}	1.00			0.98			1.00			1.00		
Pedestrian–bicycle adjustment factor for right-turn groups, f_{Rpb}			0.88			0.88			0.98			0.98
Adjusted saturation flow rate (veh/h/ln)	702	1,643	1,201	825	1,643	1,398	1,603	1,683	1,648	1,603	1,683	1,630

Notes: L = left turn; T = through; T+R = combined through and right turn.
Calculated values are based on maintaining six or more significant digits for all computed values through all calculations. These values are shown with fewer digits for presentation purposes only.

Equation 19-8 shows all the adjustment factors that might be applied in the calculation of saturation flow rate. However, when this equation is applied to a given lane group, some of the factors are not applicable (or have a value of 1.0) and can be removed from the equation. The reduced form of the saturation flow rate equation is described in the following paragraphs for several of the lane groups at the subject intersection.

For the eastbound and westbound left-turn lane groups, the adjusted saturation flow rate is calculated with the following equation.

$$s = s_p \, f_{HVg} \, f_a \, f_{Lpb}$$

The northbound and southbound left-turn lane groups operate in the protected-permitted mode. The adjusted saturation flow rate for the protected left-turn phase is calculated with the following equation.

$$s = s_o \, f_{LT} \, f_{HVg} \, f_a$$

The adjusted saturation flow rate for the permitted left-turn period is calculated with the same equation as for the eastbound and westbound left-turn lane groups.

For the through lane groups on each approach, the adjusted saturation flow rate is computed with the following equation.

$$s = s_o \, f_{HVg} \, f_a$$

For the shared-lane lane groups, the adjusted saturation flow rate is computed by using Equation 31-105. This equation is reproduced below for the eastbound shared right-turn and through lane group.

$$s_{sr} = \frac{s_{th}}{1 + P_R\left(\frac{E_R}{f_{Rpb}} - 1\right)} = \frac{1{,}438}{1 + \left(\frac{106}{186}\right)\left(\frac{1.18}{0.88} - 1\right)} = 1{,}201 \text{ veh/h/ln}$$

with

$$s_{th} = s_o \, f_{HVg} \, f_p \, f_a = 1{,}900 \times 0.96 \times 0.88 \times 0.90 = 1{,}438 \text{ veh/h/ln}$$

The calculated adjustment factors and saturation flow rates in the previous equations are based on maintaining six or more significant digits for all computed values through all calculations. These values are shown with fewer digits for presentation purposes only.

Step 5: Determine Proportion Arriving During Green

The proportion arriving during green *P* is computed using Equation 19-15. The results are shown in Exhibit 31-74. The effective green time *g* and cycle length *C* are determined by using the results from the final iteration of Step 6.

Exhibit 31-74
Example Problem 1: Proportion Arriving During Green

Data Element	Eastbound			Westbound			Northbound			Southbound		
Lane group	L	T	T+R	L	T	T+R	L	T	T+R	L	T	T+R
Phase number	2	2	2	6	6	6	3	8	8	7	4	4
Effective green time g (s)	30.0	30.0	30.0	30.0	30.0	30.0	6.2	50.0	50.0	9.8	53.6	53.6
Proportion arriving on green, P	0.29	0.29	0.29	0.29	0.29	0.29	0.06	0.49	0.49	0.10	0.53	0.53

Note: L = left turn; T = through; T+R = combined through and right turn.
Calculated values are based on maintaining six or more significant digits for all computed values through all calculations. These values are shown with fewer digits for presentation purposes only.

Step 6: Determine Signal Phase Duration

The duration of each signal phase is determined by using the procedure described in Section 2 (in the subsection titled Actuated Phase Duration). The results of this iterative process are shown in Exhibit 31-75. The resulting cycle length is 101.8 s.

Exhibit 31-75
Example Problem 1: Signal Phase Duration

Data Element	Eastbound	Westbound	Northbound		Southbound	
Phase number	2	6	3	8	7	4
Assigned movements	L+T+R	L+T+R	L	T+R	L	T+R
Phase duration D_p (s)	34.0	34.0	10.2	54.0	13.8	57.6
Maximum allowable headway *MAH* (s)	3.4	3.4	3.1	3.1	3.1	3.1
Maximum queue clearance time g_c (s)	28.7	27.2	4.1	50.0	7.6	21.2
Green extension time g_e (s)	0.0	0.4	0.2	0.0	0.3	7.8
Probability that subject phase is called, p_c	1.00	1.00	0.98	1.00	1.00	1.00
Probability of max-out, p_x	1.00	1.00	0.0	1.00	0.0	0.18
Duration of permitted left-turn green not blocked by an opposing queue, g_u (s)	11.4	17.0	32.5		0.0	

Notes: L = left turn; T = through; T+R = combined through and right turn; L+T+R = combined left, through, and right turn.
Calculated values are based on maintaining six or more significant digits for all computed values through all calculations. These values are shown with fewer digits for presentation purposes only.

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

The capacity of each through lane group and each shared-lane lane group is computed with Equation 19-16. The capacity for the permitted left-turn lane groups is computed with Equation 31-119. The latter equation is reproduced below for the eastbound left-turn lane group.

$$c_{l,e} = \frac{g_u\, s_l + 3{,}600\, n_s\, f_{ms}\, f_{sp}}{C} N_l$$

$$c_{l,e} = \frac{(11.4 \times 702) + (3{,}600 \times 2 \times 1.0 \times 1.0)}{101.8} \times 1 = 149 \text{ veh/h}$$

The capacity for the protected-permitted left-turn lane groups on the northbound and southbound approaches is computed with Equation 31-124. The results from the capacity and the volume-to-capacity ratio calculations are shown in Exhibit 31-76.

Exhibit 31-76
Example Problem 1: Capacity and Volume-to-Capacity Ratio

Data Element	Eastbound			Westbound			Northbound			Southbound		
Lane group	L	T	T+R	L	T	T+R	L	T	T+R	L	T	T+R
Phase number	2	2	2	6	6	6	3	8	8	7	4	4
Number of lanes *N* (ln)	1	1	1	1	1	1	1	1	1	1	1	1
Flow rate *v* (veh/h)	71	239	185	118	337	287	133	870	863	194	513	497
Adjusted saturation flow rate *s* (veh/h/ln)	702	1,643	1,201	825	1,643	1,398	1,603	1,683	1,648	1,603	1,683	1,630
Effective green time *g* (s)	30.0	30.0	30.0	30.0	30.0	30.0	6.2	50.0	50.0	9.8	53.6	53.6
Capacity *c* (veh/h)	149	484	354	208	484	412	328	827	809	225	887	859
Volume-to-capacity ratio *X*	0.47	0.49	0.52	0.57	0.70	0.70	0.41	1.05	1.07	0.86	0.58	0.58

Note: L = left turn; T = through; T+R = combined through and right turn.
Calculated values are based on maintaining six or more significant digits for all computed values through all calculations. These values are shown with fewer digits for presentation purposes only.

Step 8: Determine Delay

The control delay for each movement and approach, and for the intersection as a whole, is calculated with Equation 19-18. The results of the delay calculations are shown in Exhibit 31-77.

Exhibit 31-77
Example Problem 1: Control Delay

Data Element	Eastbound			Westbound			Northbound			Southbound		
Lane group	L	T	T+R	L	T	T+R	L	T	T+R	L	T	T+R
Phase number	2	2	2	6	6	6	3	8	8	7	4	4
Uniform delay d_1 (s/veh)	44.6	29.6	29.9	41.3	31.9	31.9	13.2	25.9	25.9	28.9	16.4	16.4
Incremental delay d_2 (s/veh)	0.9	0.3	0.7	2.3	3.6	4.3	0.3	46.0	50.8	3.8	0.6	0.7
Initial queue delay d_3 (s/veh)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Control delay *d* (s/veh)	45.5	29.9	30.6	43.5	35.5	36.2	13.5	72.0	76.7	32.6	17.0	17.1
Level of service	D	C	C	D	D	D	B	F	F	C	B	B
Approach delay d_A (s/veh)		32.4			37.0			70.0			19.6	
Approach LOS		C			D			E			B	
Intersection delay d_I (s/veh)	45.9											
Intersection LOS	D											

Note: L = left turn; T = through; T+R = combined through and right turn.
Calculated values are based on maintaining six or more significant digits for all computed values through all calculations. These values are shown with fewer digits for presentation purposes only.

Step 9: Determine LOS

LOS is based on the control delay. LOS values for each approach and for the entire intersection are shown in Exhibit 31-77. The determination of LOS is based on the LOS thresholds in Exhibit 19-8.

Step 10: Determine Queue Storage Ratio

The procedure for calculating the percentile back-of-queue size and queue storage ratio is described in Section 4. This procedure was used to compute the 50th percentile values for both variables. The results are shown in Exhibit 31-78.

Data Element	Eastbound			Westbound			Northbound			Southbound		
Lane group	L	T	T+R	L	T	T+R	L	T	T+R	L	T	T+R
Phase number	2	2	2	6	6	6	3	8	8	7	4	4
50th percentile back of queue $Q_{\%}$ (veh/ln)	1.8	4.8	3.8	3.0	7.6	6.6	1.4	28.9	29.4	4.9	7.7	7.5
50th percentile queue storage ratio $R_{Q\%}$	0.23	0.12	0.10	0.38	0.20	0.17	0.18	0.74	0.75	0.62	0.20	0.19

Note: L = left turn; T = through; T+R = combined through and right turn.

Exhibit 31-78
Example Problem 1: Back of Queue and Queue Storage Ratio

Queue Accumulation Polygon

The QAP is a useful way of illustrating the signal timing and performance of a signalized intersection. The evolution of the queue length during the cycle is shown in the QAP. In addition, the area of the QAP is the total uniform delay experienced by all vehicles during the cycle. The variables needed to construct the QAP for the northbound through lane group are provided in the following list. The QAP for this movement is shown in Exhibit 31-79.

- Flow rate: 870 veh/h,
- Adjusted saturation flow rate: 1,683 veh/h/ln,
- Cycle length: 101.8 s,
- Effective green time: 50.0 s,
- Effective red time: 51.8 s,
- Maximum queue clearance time: 50.0 s,
- Green extension time: 0.0 s, and
- Queue length at end of effective red: 13.4 veh/ln.

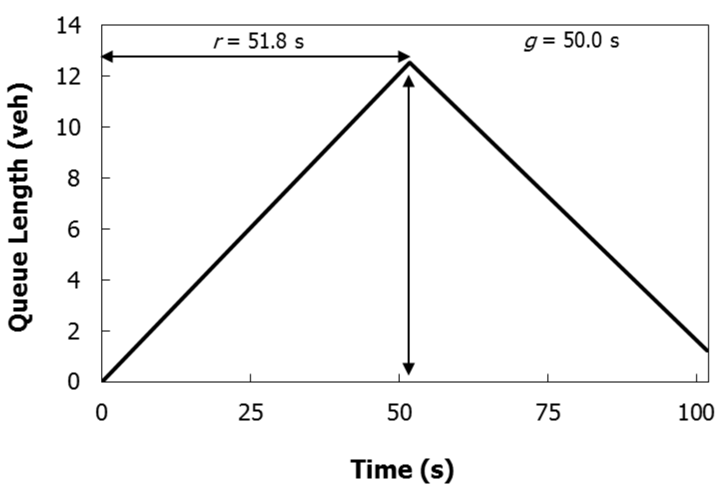


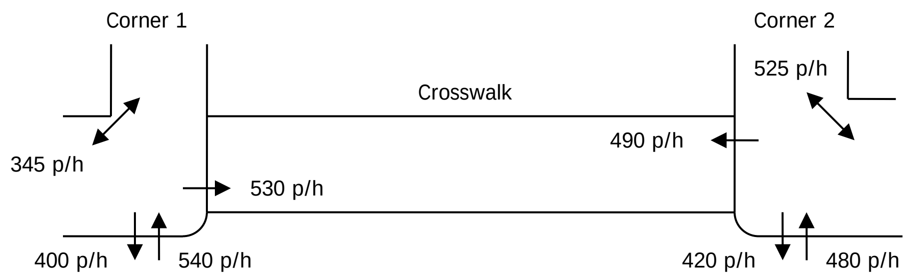
Exhibit 31-79
Example Problem 1: Queue Accumulation Polygon

EXAMPLE PROBLEM 2: PEDESTRIAN LOS

The Intersection

The pedestrian crossing of interest crosses the north leg at a signalized intersection. The north–south street is the minor street and the east–west street is the major street. The intersection serves all north–south traffic concurrently (i.e., no left-turn phases) and all east–west traffic concurrently. The signal has an 80-s cycle length. The crosswalk and intersection corners that are the subject of this example problem are shown in Exhibit 31-80.

Exhibit 31-80
Example Problem 2:
Pedestrian Flow Rates



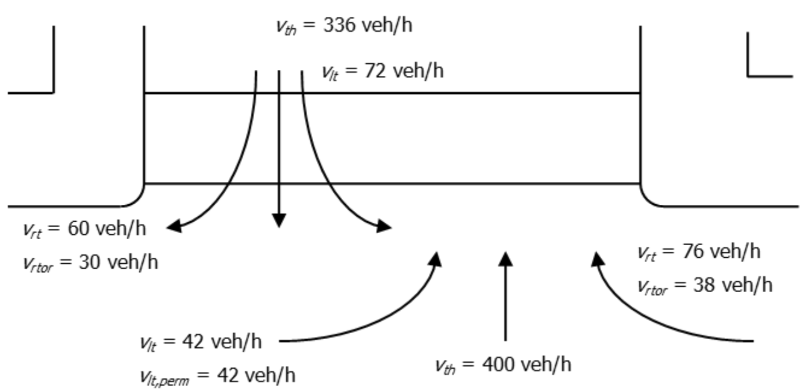
The Question

What is the pedestrian LOS for the crossing?

The Facts

Pedestrian flow rates are shown in Exhibit 31-80. Vehicular flow rates are shown in Exhibit 31-81.

Exhibit 31-81
Example Problem 2: Vehicular Demand Flow Rates



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

Major street: Phase duration, $D_{p,mj}$ = 48 s

Yellow change interval, Y_{mj} = 4 s

Red clearance interval, R_{mj} = 1 s

Walk setting, Walk_{mj} = 7 s

Pedestrian clear setting, PC_{mj} = 8 s

Four traffic lanes (no turn bays)

Minor street: Phase duration, $D_{p,mi}$ = 32 s

Yellow change interval, Y_{mi} = 4 s

Red clearance interval, R_{mi} = 1 s

Walk setting, Walk_{mi} = 7 s

Pedestrian clear setting, PC_{mi} = 13 s

Two traffic lanes (no turn bays)

85th percentile speed at a midsegment location, $S_{85,mi}$ = 35 mi/h

Corner 1: Total walkway width, $W_a = W_b$ = 16 ft

Corner radius, R = 15 ft

Corner 2: Total walkway width, $W_a = W_b$ = 18 ft

Corner radius, R = 15 ft

Other data: Effective crosswalk width, W_c = 16 ft

Crosswalk length, L_c = 28 ft

Walking speed, S_p = 4 ft/s

No right-turn channelizing islands are provided on any corner.

Pedestrian signal indications are provided for each crosswalk.

Rest-in-walk mode is not used for any phase.

Comments

On the basis of the variable notation in Exhibit 19-29, the subject crosswalk is Crosswalk C because it crosses the minor street. The outbound pedestrian flow rate v_{co} at Corner 1 equals inbound flow rate v_{ci} at Corner 2, and the inbound flow rate v_{ci} at Corner 1 equals the outbound flow rate v_{co} at Corner 2.

Outline of Solution

Pedestrian delay and the pedestrian LOS score are calculated for the crossing. Next, LOS for the crossing is determined on the basis of the computed score and the threshold values in Exhibit 19-9.

Following the determination of LOS, the example problem continues with optional steps 4 and 5. First, the circulation area is calculated for both corners. Next, the circulation area is calculated for the crosswalk. The street corner and crosswalk circulation areas are then compared with the qualitative descriptions of pedestrian space listed in Exhibit 19-34.

Computational Steps

The solution follows the steps listed in Exhibit 19-30 of Chapter 19.

Step 1: Determine Pedestrian Delay

Because pedestrian signal indications are provided and rest-in-walk is not enabled, the effective walk time for the phase serving the major street is computed with Equation 19-51.

$$g_{\text{Walk},mj} = \text{Walk}_{mj} + 4.0 = 7.0 + 4.0 = 11.0 \text{ s}$$

The pedestrian delay is calculated by using Equation 19-54.

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

$$d_p = \frac{(80 - 11)^2}{2(80)} = 29.8 \text{ s/p}$$

Step 2: Determine Pedestrian LOS Score for Intersection

The number of vehicles traveling on the minor street during a 15-min period is computed by using Equation 19-60.

$$n_{15,mi} = \frac{0.25}{N_c} \sum v_i$$

$$n_{15,mi} = \frac{0.25}{2}(72 + 336 + 60 + 42 + 400 + 76) = 123.3 \text{ veh/ln}$$

The cross-section adjustment factor is calculated by using Equation 19-56.

$$F_w = 0.681(N_c)^{0.514}$$

$$F_w = 0.681(2)^{0.514} = 0.972$$

The motorized vehicle adjustment factor is computed with Equation 19-57.

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

$$F_v = 0.00569\left(\frac{30 + 42}{4}\right) - (0)(0.0027(123.3) - 0.1946) = 0.102$$

The motorized vehicle speed adjustment factor is then computed with Equation 19-58.

$$F_s = 0.00013\, n_{15,mi}\, S_{85,mi}$$

$$F_s = 0.00013(123.3)(35) = 0.561$$

The pedestrian delay adjustment factor is calculated with Equation 19-59.

$$F_{\text{delay}} = 0.0401 \ln\left(d_{p,c}\right)$$

$$F_{\text{delay}} = 0.0401 \ln(29.8) = 0.136$$

The pedestrian LOS score for the intersection $I_{p,int}$ is then computed with Equation 19-55.

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

$$I_{p,int} = 0.5997 + 0.972 + 0.102 + 0.561 + 0.136 = 2.37$$

Step 3: Determine LOS

According to Exhibit 19-9, the crosswalk operates at LOS B.

Step 4: Determine Street Corner Circulation Area

A. Compute Available Time–Space

For Corner 1, the available time–space is computed with Equation 19-61.

$$TS_{\text{corner}} = C(W_a\, W_b - 0.215\, R^2)$$
$$TS_{\text{corner}} = 80[16 \times 16 - 0.215(15)^2]$$
$$TS_{\text{corner}} = 16{,}610 \text{ ft}^2\text{-s}$$

B. Compute Holding-Area Waiting Time

The number of pedestrians arriving at the corner during each cycle to cross the minor street is computed with Equation 19-63.

$$N_{co} = \frac{v_{co}}{3{,}600} C$$
$$N_{co} = \frac{530}{3{,}600}(80) = 11.8 \text{ p}$$

The total time spent by pedestrians waiting to cross the minor street during one cycle is then calculated with Equation 19-62. The effective walk time $g_{\text{Walk},mj}$ was determined in Step 1.

$$Q_{tco} = \frac{N_{co}\left(C - g_{\text{Walk,mj}}\right)^2}{2\,C}$$
$$Q_{tco} = \frac{(11.8)(80 - 11)^2}{2(80)} = 350.5 \text{ p-s}$$

By the same procedure, the total time spent by pedestrians waiting to cross the major street during one cycle (Q_{tdo}) is found to be 264.5 p-s.

C. Compute Circulation Time–Space

The circulation time–space is found by using Equation 19-64.

$$TS_c = TS_{\text{corner}} - [5.0(Q_{tdo} + Q_{tco})]$$
$$TS_c = 16{,}610 - [5.0(350.5 + 264.5)] = 13{,}535 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Corner Circulation Area

The total number of circulating pedestrians is computed with Equation 19-66.

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600} C$$
$$N_{tot} = \frac{490 + 530 + 540 + 400 + 345}{3{,}600}(80) = 51.2 \text{ p}$$

Finally, the corner circulation area per pedestrian is calculated with Equation 19-65.

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

$$M_{\text{corner}} = \frac{13{,}535}{4.0(51.2)} = 66.1 \text{ ft}^2/\text{p}$$

By following the same procedure, the corner circulation area per pedestrian for Corner 2 is found to be 87.6 ft^2/p. According to the qualitative descriptions provided in Exhibit 19-34, pedestrians at both corners will have the ability to move in the desired path without needing to alter their movements to avoid conflicts.

Step 5: Determine Crosswalk Circulation Area

The analysis conducted in this step describes the circulation area for pedestrians in the subject crosswalk.

A. Establish Walking Speed

As given in the subsection titled The Facts, the average walking speed is determined to be 4.0 ft/s.

B. Compute Available Time–Space

Rest-in-walk is not enabled, so the pedestrian service time g_{ped} is estimated to equal the sum of the walk and pedestrian clear settings. The time–space available in the crosswalk is found with Equation 19-67.

$$TS_{cw} = L_c\, W_c\, g_{\text{Walk},mj}$$

$$TS_{cw} = (28)(16)(11) = 4{,}928 \text{ ft}^2\text{-s}$$

C. Compute Effective Available Time–Space

The number of turning vehicles during the walk and pedestrian clear intervals is calculated with Equation 19-68.

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

$$N_{tv} = \frac{42 + 76 - 38}{3{,}600}(80) = 1.8 \text{ veh}$$

The time–space occupied by turning vehicles can then be computed with Equation 19-69.

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

$$TS_{tv} = 40(1.8)(16) = 1{,}138 \text{ ft}^2\text{-s}$$

The effective available crosswalk time–space TS^*_{cw} is found by subtracting the total available crosswalk time–space TS_{cw} from the time–space occupied by turning vehicles, as shown by Equation 19-68.

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

$$TS^*_{cw} = 4{,}928 - 1{,}138 = 3{,}970 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Service Time

The number of pedestrians exiting the curb when the WALK indication is presented is computed by using Equation 19-73.

$$N_{ped,co} = N_{co} \frac{C - g_{\text{Walk},mj}}{C}$$

$$N_{ped,co} = (11.8) \frac{80 - 11}{80} = 10.2 \text{ p}$$

Because the crosswalk width is greater than 10 ft, the pedestrian service time is computed by using Equation 19-71.

$$t_{ps,co} = 3.2 + \frac{L_c}{S_p} + 2.7 \frac{N_{ped,co}}{W_c}$$

$$t_{ps,co} = 3.2 + \frac{28}{4.0} + (2.7) \frac{10.2}{16} = 11.9 \text{ s}$$

The other travel direction in the crosswalk is analyzed next. The number of pedestrians arriving at Corner 1 each cycle by crossing the minor street is computed by using Equation 19-75.

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

$$N_{ci} = \frac{490}{3{,}600} (80) = 10.9 \text{ p}$$

The sequence of calculations is repeated for this second travel direction in the subject crosswalk to indicate that $N_{ped,ci}$ is equal to 9.4 p and $t_{ps,ci}$ is 11.8.

E. Compute Crosswalk Occupancy Time

The crosswalk occupancy time for the crosswalk is computed by using Equation 19-74.

$$T_{occ} = t_{ps,co} N_{co} + t_{ps,ci} N_{ci}$$

$$T_{occ} = 11.9(11.8) + 11.8(10.9) = 268.6 \text{ p-s}$$

F. Compute Pedestrian Crosswalk Circulation Area

Finally, the crosswalk circulation area per pedestrian for the crosswalk is computed by using Equation 19-76.

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

$$M_{cw} = \frac{3{,}790}{268.6} = 14.1 \text{ ft}^2/\text{p}$$

The crosswalk circulation area is found to be 14.1 ft^2/p. According to the qualitative descriptions provided in Exhibit 19-28, pedestrians will find their walking speed is restricted, with very limited ability to pass slower pedestrians. Improvements to the crosswalk should be considered and may include a wider crosswalk or a longer walk interval.

Discussion

The crosswalk was found to operate at LOS B in Step 3. It was determined in Step 4 that the pedestrians at both corners have adequate space to allow freedom of movement. However, crosswalk circulation area was found to be restricted in Step 5 and improvements are probably justified. Moreover, the pedestrian delay computed in Step 1 was found to be slightly less than 30 s/p. With this much delay, some pedestrians may not comply with the signal indication.

EXAMPLE PROBLEM 3: BICYCLE LOS

The Intersection

A 5-ft-wide bicycle lane is provided at a signalized intersection.

The Question

What is the LOS of this bicycle lane?

The Facts

Saturation flow rate for bicycles = 2,000 bicycles/h

Effective green time = 48 s

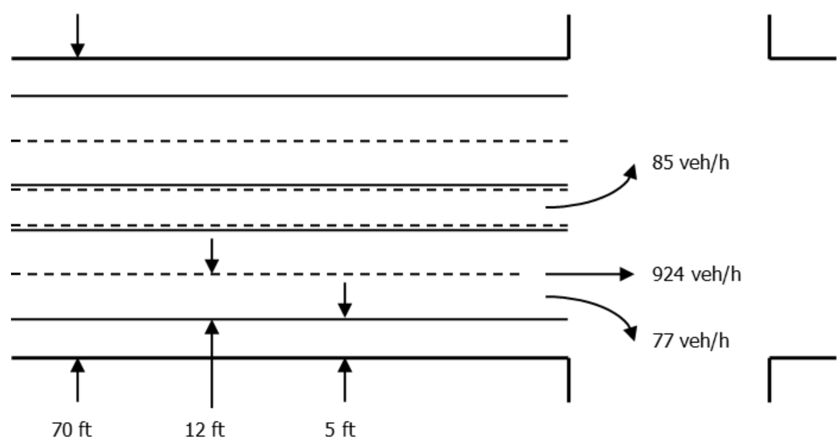
Cycle length = 120 s

Bicycle flow rate = 120 bicycles/h

No on-street parking

The vehicular flow rates and street cross-section element widths are as shown in Exhibit 31-82.

Exhibit 31-82
Example Problem 3: Vehicular Demand Flow Rates and Cross-Section Element Widths



Outline of Solution

Bicycle delay and the bicycle LOS score are computed. LOS is then determined on the basis of the computed score and the threshold values in Exhibit 19-9.

Computational Steps

The solution follows the steps listed in Exhibit 19-40 of Chapter 19.

Step 1: Determine Bicycle Delay

A. Compute Bicycle Lane Capacity

The capacity of the bicycle lane is calculated with Equation 19-106.

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

$$c_b = (2{,}000)\frac{48}{120} = 800 \text{ bicycles/h}$$

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 19-107.

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

$$d_b = \frac{0.5(120)(1-48/120)^2}{1-\min\left(\frac{120}{800},1.0\right)\times\frac{48}{120}} = 23.0 \text{ s/bicycle}$$

Step 2: Determine Bicycle LOS Score for Intersection

As shown in Exhibit 31-82, the total width of the outside through lane, bicycle lane, and paved shoulder W_t is 17 ft (= 12 + 5 + 0 + 0). There is no on-street parking. The cross-section adjustment factor can then be calculated with Equation 19-109.

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

$$F_w = 0.0153(70) - 0.2144(17) = -2.57$$

The motor-vehicle volume adjustment factor must be calculated by using Equation 19-110.

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

$$F_v = 0.0066 \frac{85 + 924 + 77}{4(2)} = 0.90$$

The bicycle LOS score can then be computed with Equation 19-108.

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

$$I_{b,int} = 4.1324 - 2.57 + 0.90 = 2.45$$

Step 3: Determine LOS

According to Exhibit 19-9, the bicycle lane will operate at LOS B through the signalized intersection.

Discussion

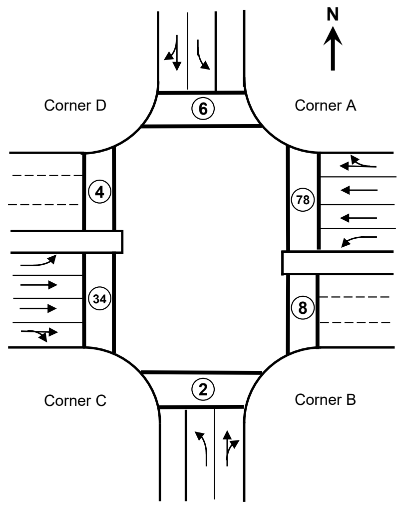
The bicycle lane was found to operate at LOS B. The bicycle delay was found to be 23.0 s/bicycle, which is low enough that most bicyclists are not likely to be impatient. However, if the signal timing at the intersection were to be changed, the bicycle delay would need to be computed again to verify that it does not rise above 30 s/bicycle.

EXAMPLE PROBLEM 4: PEDESTRIAN DELAY WITH TWO-STAGE CROSSING OF ONE INTERSECTION LEG

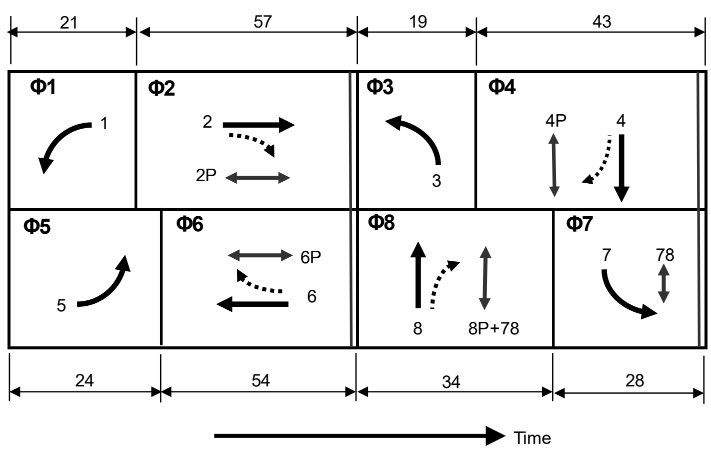
The Intersection

The pedestrian crosswalk of interest is on the east leg of an intersection. This intersection is shown in Exhibit 31-83(a) where north is toward the top of the figure. The pedestrian movement of interest travels north, from corner B to corner A. A two-stage crossing is provided for this leg of the intersection. Signal heads are provided for all pedestrian movements.

Exhibit 31-83
Example Problem 4: Intersection Geometry and Signal Phase Sequence



(a) Intersection geometry



(b) Signal phase sequence

The east–west street is the major street. The major street traffic signals provide coordination for the through movements using a 140-s cycle length. The minor movements are actuated. Rest-in-walk is not used for any phases. The duration of each phase is shown in Exhibit 31-83(b). The Walk intervals are set at 5.0 s and they start at the same time as the associated phase (i.e., they do not lead or lag the phase). The distance crossed for crosswalk section 8 is 40 ft. The median on the major street (at the location of pedestrian storage) is 16 ft wide.

The Question

The analyst desires to estimate the delay to the northbound pedestrian movement on the east leg of the intersection.

Computational Steps

The solution follows the steps of the "Crossing One Intersection Leg in Two Stages" procedure. The first stage of the crossing occurs in crosswalk section 8. It is served by phase 8 so Phase X is phase 8 (i.e., X = 8). The second stage of the crossing occurs in crosswalk section 78. It is served by phases 7 and 8. Phase 7 is shown to occur next (after 8) so Phase Y is phase 7 (i.e., Y = 7).

Step 1. Determine the Effective Walk Time

The subject phases are actuated and rest-in-walk is not enabled so the effective walk time is determined by Equation 19-51.

$$g_{\text{Walk},i} = \text{Walk}_i + 4.0 = 5.0 + 4.0 = 9.0 \text{ s}$$

Step 2. Determine Crossing Time during First Stage

The local pedestrian population is about 30 percent elderly so an average pedestrian crossing speed of 3.3 ft/s is used for the analysis. Equation 19-88 is used to compute the time for pedestrians to travel the 56-ft distance (= 40 + 16) from corner B to the far side of the median.

$$t_X = \frac{L_X}{S_p} = \frac{56}{3.3} = 17.0 \text{ s}$$

Step 3. Determine the Start of the Walk Intervals

The relative start time of the Walk intervals for Phase X and Phase Y are determined by inspection of Exhibit 31-83(b). For Phase X (i.e., phase 8), the relative start time $T_{\text{Walk},X}$ is 78 s (= 21 + 57). For Phase Y (i.e., phase 7), the relative start time $T_{\text{Walk},Y}$ is 112 s (= 21 + 57 + 34).

Step 4. Compute Delay for First-Stage Crossing

The delay for the first-stage crossing is computed using Equation 19-78.

$$d_{p,1} = \frac{\left(C - g_{\text{Walk},X}\right)^2}{2\,C} = \frac{(140 - 9.0)^2}{2\ \times 140} = 61.3 \text{ s/p}$$

Step 5. Compute Delay for Second-Stage Crossing Given Arrival is during Don't Walk

The time between the Walk intervals for Phases X and Y is computed using Equation 19-79.

$$t_{YX} = \text{Modulo}\left(T_{\text{Walk},Y} - T_{\text{Walk},X}, C\right) = \text{Modulo}(112 - 78, 140) = 34.0 \text{ s}$$

The waiting time on the median (for those pedestrians that reach the median during a Don't Walk indication) is computed using Equation 19-81.

$$t = \text{Modulo}(t_{YX} - t_X, C) = \text{Modulo}(34.0 - 17.0, 140) = 17.0 \text{ s}$$

The delay incurred by pedestrians waiting *on the median* that arrived at the first corner during a Don't Walk indication $d_{2,DW1}$ is computed using Equation 19-80.

$$d_{2,DW1} = \begin{cases} t & \text{if } t < C - g_{\text{Walk},Y} \\ 0 & \text{if } t \geq C - g_{\text{Walk},Y} \end{cases}$$

Because the value of t is less than $C - g_{\text{Walk},Y}$ (i.e., 17 < 140 – 9), the delay $d_{2,DW1}$ is equal to t, or 17.0 s/p.

Step 6. Compute Delay for Second-Stage Crossing Given Arrival is during Walk

Because t is greater than $g_{\text{Walk},X}$ (i.e., 17 > 9), Equation 19-84 is used to compute the delay on the median for stage 2.

$$d_{2,W1} = \begin{cases} t - 0.5\, g_{\text{Walk},X} & \text{if } \left(t + g_{\text{Walk},Y}\right) < C \\ \dfrac{0.5\, b^2 + b\left(t - g_{\text{Walk},X}\right)}{g_{\text{Walk},X}} & \text{if } C \leq \left(t + g_{\text{Walk},Y}\right) \leq \left(C + g_{\text{Walk},X}\right) \\ 0 & \text{if } \left(t + g_{\text{Walk},Y}\right) > \left(C + g_{\text{Walk},X}\right) \end{cases}$$

Because $t + g_{\text{Walk},Y}$ is less than C (i.e., 17 + 9 < 140), the first part of this equation is used to compute the desired delay value.

$$d_{2,W1} = t - 0.5\, g_{\text{Walk},X} = 17 - 0.5 \times 9 = 12.5 \text{ s/p}$$

Step 7. Compute Delay for Two-Stage Crossing

The proportion of arrivals during the Don't Walk indication at the corner P_{DW1} is computed using Equation 19-87.

$$P_{DW1} = \frac{\left(C - g_{\text{Walk},X}\right)}{C} = \frac{(140 - 9)}{140} = 0.936$$

Equation 19-86 is then used to compute the delay for the two-stage crossing.

$$d_p = d_{p,1} + \left[d_{2,DW1} P_{DW1} + d_{2,W1}(1 - P_{DW1})\right]$$

$$d_p = 61.3 + [(17.0 \times 0.936) + 12.5(1 - 0.936)]$$

$$d_p = 78 \text{ s/p}$$

Discussion

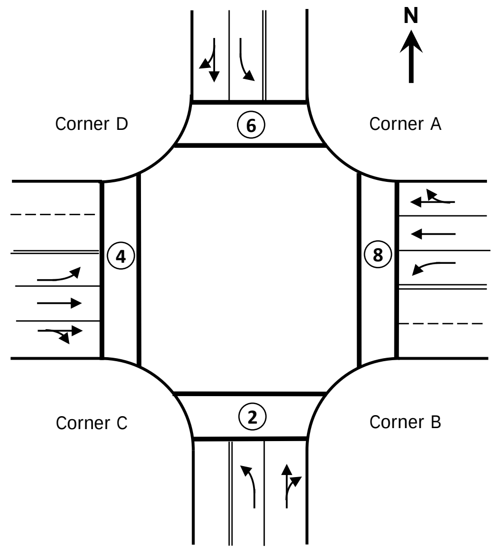
Unlike a one-stage crossing, the delay crossing in the one direction will generally be different than the delay crossing in the opposite direction. In this case, using the same inputs given above, plus a crosswalk length of 52 ft for crosswalk section 78, the average delay traveling from Corner A to Corner B is calculated to be 147 s.

EXAMPLE PROBLEM 5: PEDESTRIAN DELAY WITH TWO-STAGE CROSSING OF TWO INTERSECTION LEGS

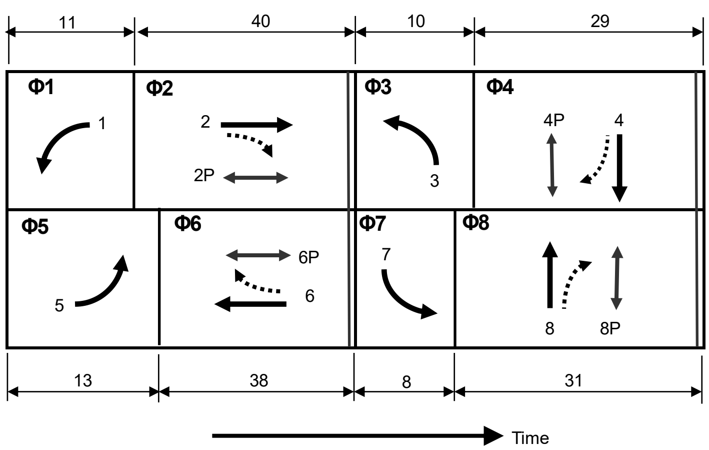
The Intersection

The pedestrian crosswalks of interest are on the south and west legs of an intersection. This intersection is shown in Exhibit 31-84(a) where north is toward the top of the figure. The pedestrian movement of interest travels clockwise from corner B to corner C and then to corner D. Signal heads are provided for all pedestrian movements.

Exhibit 31-84
Example Problem 5:
Intersection Geometry and
Signal Phase Sequence



(a) Intersection geometry



(b) Signal phase sequence

The east-west street is the major street. The major street traffic signals provide coordination for the through movements using a 90-s cycle length. The minor movements are actuated. Rest-in-walk is not used for any phases. The duration of each phase is shown in Exhibit 31-84(b). The Walk intervals are set at 5.0 s and they start at the same time as the associated phase (i.e., they do not lead or lag the phase). The distance crossed for crosswalk 2 is 38 ft.

The Question

The analyst desires to estimate the delay to pedestrians traveling from corner B to corner D by crossing the south leg and then the west leg.

Computational Steps

The solution follows the steps of the "Crossing Two Intersection Legs in Two Stages" procedure. The first stage of the crossing occurs in crosswalk 2. It is served by phase 2 so Phase X is phase 2 (i.e., X = 2). The second stage of the crossing occurs in crosswalk 4. It is served by phase 4 so Phase Y is phase 4 (i.e., Y = 4). Finally, if the pedestrian decided to cross in the other direction around the intersection (i.e., counterclockwise), the first phase to serve this travel direction is phase 8 so Phase Z is phase 8 (i.e., Z = 8).

Step 1. Determine the Effective Walk Time

The subject phases are actuated and rest-in-walk is not enabled so the effective walk time for Phase X and Phase Z is determined by Equation 19-51.

$$g_{\text{Walk},i} = \text{Walk}_i + 4.0 = 5.0 + 4.0 = 9.0 \text{ s}$$

Step 2. Determine Crossing Time during First Phase

The local pedestrian population is about 30 percent elderly so an average pedestrian crossing speed of 3.3 ft/s is used for the analysis. Equation 19-88 is used to compute the time for pedestrians to travel the 38-ft distance from corner B to corner C.

$$t_X = \frac{L_X}{S_p} = \frac{38}{3.3} = 11.5 \text{ s}$$

Step 3. Determine the Start of the Walk Intervals

The relative start time of the Walk intervals for Phase X, Phase Y, and Phase Z are determined by inspection of Exhibit 31-84(b). For Phase X (i.e., phase 2), the relative start time $T_{\text{Walk},X}$ is 11 s. For Phase Y (i.e., phase 4), the relative start time $T_{\text{Walk},Y}$ is 61 s (= 11 + 40 + 10). For Phase Z (i.e., phase 8), the relative start time $T_{\text{Walk},Z}$ is 59 s (= 11 + 40 + 8).

Step 4. Compute Delay for First-Stage Crossing

The end of the effective walk time for Phase X is computed using Equation 19-88.

$$T_X = \text{Modulo}\left(T_{\text{Walk},X} + g_{\text{Walk},X}, C\right) = \text{Modulo}(11 + 9, 90) = 20 \text{ s}$$

Similarly, the end of the effective walk time for Phase Z is computed using Equation 19-89.

$$T_Z = \text{Modulo}\left(T_{\text{Walk},Z} + g_{\text{Walk},Z}, C\right) = \text{Modulo}(59 + 9, 90) = 68 \text{ s}$$

The time between the end of effective walk time for Phase Z and the start of effective walk time for Phase X is computed using Equation 19-91.

$$t_{XZ} = \text{Modulo}(T_X - T_Z, C) = \text{Modulo}(20 - 68, 90) = 20 - 68 + 90 = 42 \text{ s}$$

Finally, the delay for the first stage crossing is computed using Equation 19-90.

$$d_{p,1} = \frac{\left(t_{XZ} - g_{\text{Walk},X}\right)^2}{2\ t_{XZ}} = \frac{(42 - 9)^2}{2\ \times 42} = 13.0 \text{ s/p}$$

Step 5. Compute Delay for Entire Diagonal Crossing

Equation 19-93 is used to determine the time between arrival at corner B and departure from the second corner.

$$t_d = \begin{cases} T_{\text{Walk},Y} - \dfrac{T_X + T_Z}{2} & \text{if } T_{\text{Walk},Y} \geq T_X \geq T_Z \\ T_{\text{Walk},Y} - \dfrac{T_X + T_Z - C}{2} & \text{if } T_X < T_Z \\ T_{\text{Walk},Y} - \dfrac{T_X + T_Z}{2} + C & \text{if } T_X \geq T_Z \geq T_{\text{Walk},Y} \end{cases}$$

Because the relative end time of the effective walk period for Phase X T_X is less than that for Phase Z T_Z (i.e., 20 < 68), the second part of Equation 19-93 is used to compute the desired time interval t_d.

$$t_d = T_{Walk,Y} - \frac{T_X + T_Z - C}{2} = 61 - \frac{20 + 68 - 90}{2} = 62 \text{ s}$$

Finally, the diagonal crossing delay is computed using Equation 19-92.

$$d_p = t_d - t_X = 62.0 - 11.5 = 50.5 \text{ s/p}$$

Step 6. Compute Delay for Second Stage Crossing

The delay for the second-stage crossing $d_{p,2}$ is computed using Equation 19-94.

$$d_{p,2} = d_p - d_{p,1} = 50.5 - 13.0 = 37.5 \text{ s/p}$$

Discussion

Reasonably good signal compliance can be expected for the first-stage crossing, based on the average delay of 13.0 s. However, the second-stage delay exceeds 30 s and some pedestrians may not comply with the signal indications.

10. REFERENCES

Many of these references are available in the Technical Reference Library in Volume 4.

1. Zegeer, C., K. Opiela, and M. Cynecki. *Pedestrian Signalization Alternatives*. Report FHWA/RD-83/102. Federal Highway Administration, Washington, D.C., 1983.
2. Lieberman, E. B. Determining the Lateral Deployment of Traffic on an Approach to an Intersection. In *Transportation Research Record 772*, Transportation Research Board, National Research Council, Washington, D.C., 1980, pp. 1–5.
3. Bonneson, J. Lane Volume and Saturation Flow Rate for a Multilane Intersection Approach. *Journal of Transportation Engineering*, Vol. 124, No. 3, 1998, pp. 240–246.
4. Bonneson, J. A., and P. T. McCoy. *NCHRP Report 395: Capacity and Operational Effects of Midblock Left-Turn Lanes*. Transportation Research Board, National Research Council, Washington, D.C., 1997.
5. Teply, S. Accuracy of Delay Surveys at Signalized Intersections. In *Transportation Research Record 1225*, Transportation Research Board, National Research Council, Washington, D.C., 1989, pp. 24–32.
6. *Transportation Research Circular 212: Interim Materials on Highway Capacity*. Transportation Research Board, National Research Council, Washington, D.C., 1980.
7. Pline, J. L. (ed.). *Traffic Engineering Handbook*, 5th ed. ITE, Washington, D.C., 1999.
8. Reilly, W. R., and C. C. Gardner. Technique for Measuring Delay at Intersections. In *Transportation Research Record 644*, Transportation Research Board, National Research Council, Washington, D.C., 1977, pp. 1–7.
9. Powell, J. L. Field Measurement of Signalized Intersection Delay for 1997 Update of the *Highway Capacity Manual*. In *Transportation Research Record 1646*, Transportation Research Board, National Research Council, Washington, D.C., 1998, pp. 79–86.

Enhancing Pedestrian Volume Estimation and
Developing HCM Pedestrian Methodologies
for Safe and Sustainable Communities

Chapter 32: STOP-Controlled Intersections: Supplemental

Version 6.1 Final Draft

Prepared for:
NCHRP Project 17-87

June 26, 2020

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This chapter contains revisions to TWSC Example Problem 2 in Section 3 that incorporate changes to the pedestrian LOS methodology in Chapter 20. These revisions were developed through NCHRP Project 17-87.

CHAPTER 32
STOP-CONTROLLED INTERSECTIONS: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

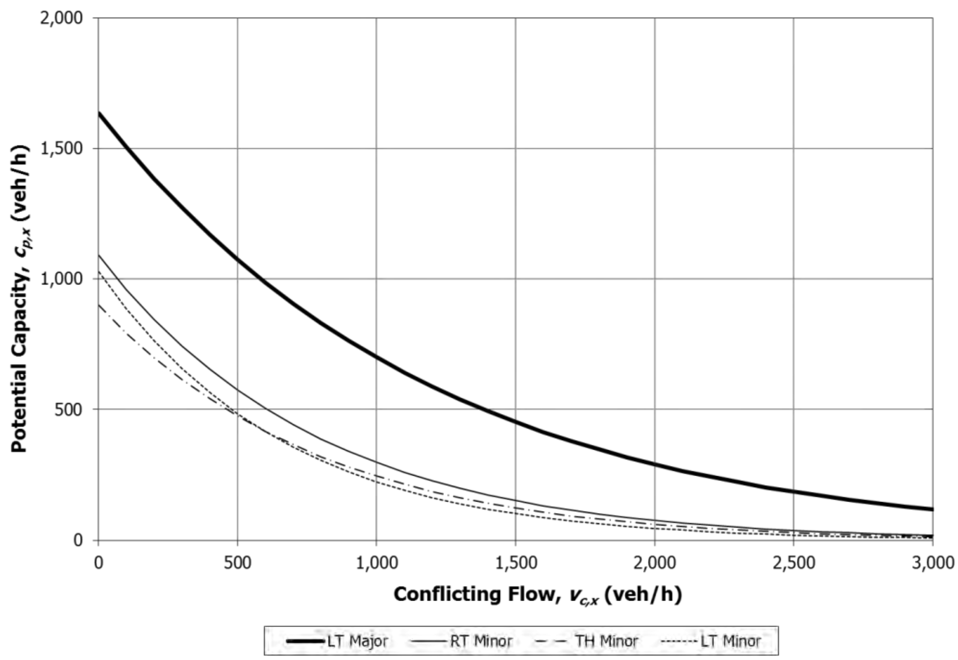
Chapter 32 is the supplemental chapter for Chapter 20, Two-Way STOP-Controlled Intersections, and Chapter 21, All-Way STOP-Controlled Intersections, which are found in Volume 3 of the *Highway Capacity Manual*. This chapter provides supplemental material on (*a*) determining the potential capacity of two-way STOP-controlled (TWSC) intersections and (*b*) identifying the 512 combinations of degree-of-conflict cases for all-way STOP-controlled (AWSC) intersections with three-lane approaches. The chapter also provides example problems demonstrating the application of the TWSC and AWSC methodologies.

VOLUME 4: APPLICATIONS GUIDE

2. TWSC POTENTIAL CAPACITY

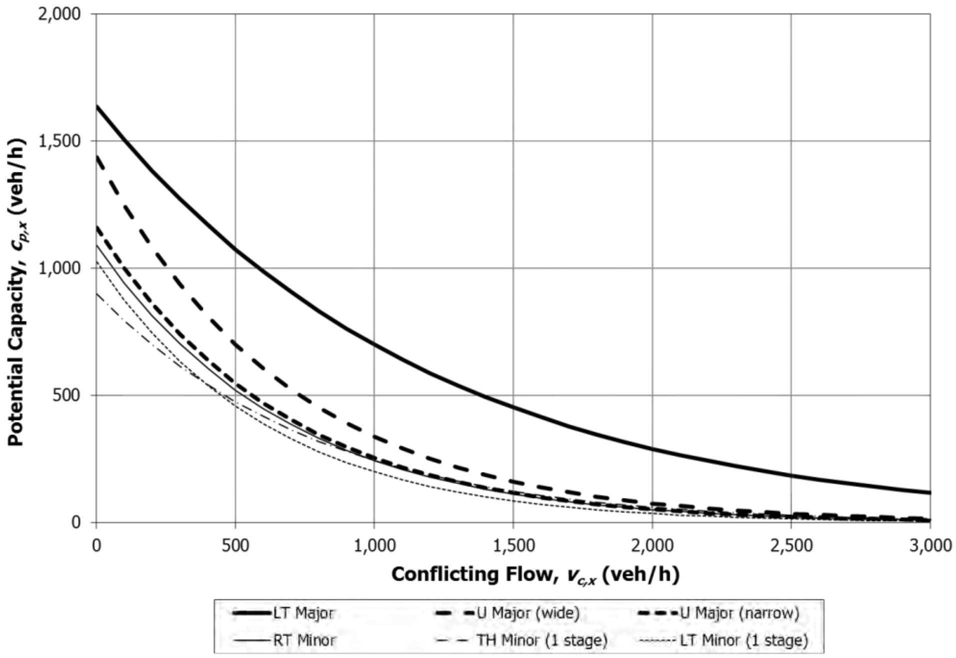
The gap acceptance model to estimate potential capacity (presented in Chapter 20, Equation 20-32) can be plotted for each of the non–Rank 1 movements by using values of critical headway and follow-up headway from Chapter 20 (Exhibit 20-12 and Exhibit 20-13, respectively). These graphs are presented in Exhibit 32-1, Exhibit 32-2, and Exhibit 32-3 for a major street with two lanes, four lanes, and six lanes, respectively. The potential capacity is expressed as vehicles per hour. The exhibits indicate the potential capacity is a function of the conflicting flow rate $v_{c,x}$ expressed as an hourly rate, as well as the type of minor-street movement.

Exhibit 32-1
Potential Capacity $c_{p,x}$ for Two-Lane Major Streets



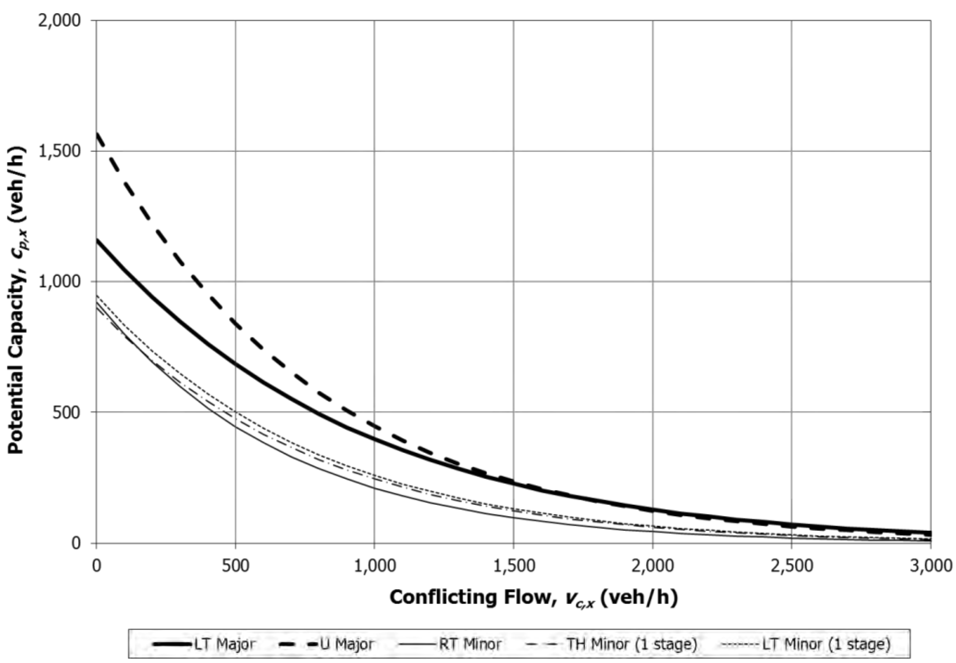
Note: LT = left turn, RT = right turn, and TH = through.

Exhibit 32-2
Potential Capacity $c_{p,x}$ for Four-Lane Major Streets



Note: LT = left turn, U = U-turn, RT = right turn, and TH = through.

Exhibit 32-3
Potential Capacity $c_{p,x}$ for Six-Lane Major Streets



Note: LT = left turn, U = U-turn, RT = right turn, and TH = through.

3. TWSC EXAMPLE PROBLEMS

This section provides example problems for use of the TWSC methodology. Exhibit 32-4 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operations analysis level in terms of the calculations, except that default values are used when available.

Exhibit 32-4
TWSC Example Problems

Problem Number	Description	Analysis Level
1	TWSC at an intersection with three legs	Operational
2	Pedestrian crossing at a TWSC intersection	Operational
3	TWSC intersection with flared approaches and median storage	Operational
4	TWSC intersection within a signalized urban street segment	Operational
5	TWSC intersection on a six-lane street with U-turns and pedestrians	Operational

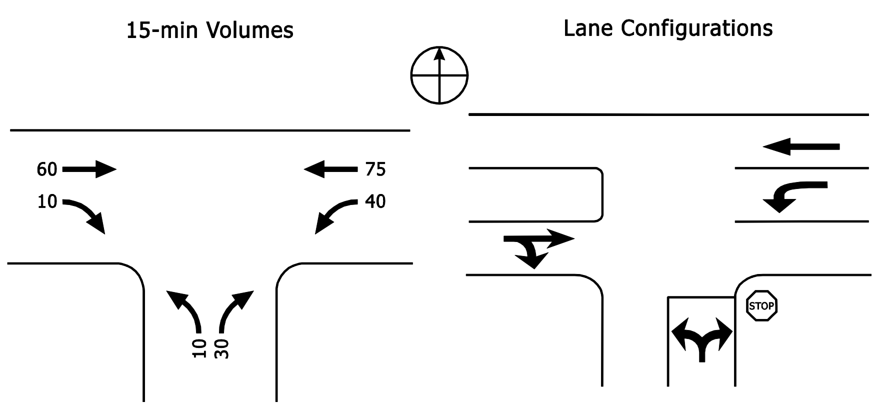
TWSC EXAMPLE PROBLEM 1: TWSC AT AN INTERSECTION WITH THREE LEGS

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- T-intersection,
- Major street with one lane in each direction,
- Minor street with one lane in each direction and STOP-controlled on the minor-street approach,
- Level grade on all approaches,
- Percentage heavy vehicles on all approaches = 10%,
- No other unique geometric considerations or upstream signal considerations,
- No pedestrians,
- Length of analysis period = 0.25 h, and
- Volumes during the peak 15-min period and lane configurations as shown in Exhibit 32-5.

Exhibit 32-5
TWSC Example Problem 1: 15-min Volumes and Lane Configurations



Comments

All input parameters are known, so no default values are needed or used.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Because peak 15-min volumes have been provided, each volume is multiplied by four to determine a peak 15-min flow rate (in vehicle per hour) for each movement. These values, along with the associated movement numbers, are shown in Exhibit 32-6.

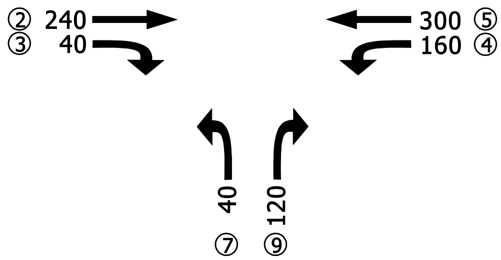


Exhibit 32-6
TWSC Example Problem 1: Movement Numbers and Calculation of Peak 15-min Flow Rates

Step 3: Compute Conflicting Flow Rates

The conflicting flow rates for each minor movement at the intersection are computed according to Equation 20-3, Equation 20-4, Equation 20-18, and Equation 20-24. The conflicting flow for the major-street left-turn $v_{c,4}$ is

$$v_{c,4} = v_2 + v_3 + v_{15}$$
$$v_{c,4} = 240 + 40 + 0 = 280 \text{ veh/h}$$

The conflicting flow for the minor-street right-turn movement $v_{c,9}$ is

$$v_{c,9} = v_2 + 0.5v_3 + v_{14} + v_{15}$$
$$v_{c,9} = 240 + 0.5(40) + 0 + 0 = 260 \text{ veh/h}$$

Finally, the conflicting flow for the minor-street left-turn movement $v_{c,7}$ is computed. Because two-stage gap acceptance is not present at this intersection, the conflicting flow rates shown in Stage I (Equation 20-18) and Stage II (Equation 20-24) are added together and considered as one conflicting flow rate. The conflicting flow for $v_{c,7}$ is computed as follows:

$$v_{c,7} = 2v_1 + v_2 + 0.5v_3 + v_{15} + 2v_4 + v_5 + 0.5v_6 + 0.5v_{12} + 0.5v_{11} + v_{13}$$
$$v_{c,7} = 2(0) + 240 + 0.5(40) + 0 + 2(160) + 300 + 0.5(0) + 0.5(0) + 0.5(0) + 0 = 880 \text{ veh/h}$$

Step 4: Determine Critical Headways and Follow-Up Headways

The critical headway for each minor movement is computed beginning with the base critical headway given in Exhibit 20-12. The base critical headway for each movement is then adjusted according to Equation 20-30. The critical headway for the major-street left-turn movement $t_{c,4}$ is computed as follows:

$$t_{c,4} = t_{c,\text{base}} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$
$$t_{c,4} = 4.1 + 1.0(0.1) + 0(0) - 0 = 4.2 \text{ s}$$

Similarly, the critical headway for the minor-street right-turn movement $t_{c,9}$ is

$$t_{c,9} = 6.2 + 1.0(0.1) + 0.1(0) - 0 = 6.3 \text{ s}$$

Finally, the critical headway for the minor-street left-turn movement $t_{c,7}$ is

$$t_{c,7} = 7.1 + 1.0(0.1) + 0.2(0) - 0.7 = 6.5 \text{ s}$$

The follow-up headway for each minor movement is computed beginning with the base follow-up headway given in Exhibit 20-13. The base follow-up headway for each movement is then adjusted according to Equation 20-31. The follow-up headway for the major-street left-turn movement $t_{f,4}$ is computed as follows:

$$t_{f,4} = t_{f,base} + t_{f,HV}P_{HV}$$

$$t_{f,4} = 2.2 + 0.9(0.1) = 2.29 \text{ s}$$

Similarly, the follow-up headway for the minor-street right-turn movement $t_{f,9}$ is

$$t_{f,9} = 3.3 + 0.9(0.1) = 3.39 \text{ s}$$

Finally, the follow-up headway for the minor-street left-turn movement $t_{f,7}$ is

$$t_{f,7} = 3.5 + 0.9(0.1) = 3.59 \text{ s}$$

Step 5: Compute Potential Capacities

The computation of a potential capacity for each movement provides the analyst with a definition of capacity under the assumed base conditions. The potential capacity will be adjusted in later steps to estimate the movement capacity for each movement. The potential capacity for each movement is a function of the conflicting flow rate, critical headway, and follow-up headway computed in the previous steps. The potential capacity for the major-street left-turn movement $c_{p,4}$ is computed as follows from Equation 20-32:

$$c_{p,4} = v_{c,4}\frac{e^{-v_{c,4}t_{c,4}/3{,}600}}{1 - e^{-v_{c,4}t_{f,4}/3{,}600}}$$

$$c_{p,4} = 280\frac{e^{-(280)(4.2)/3{,}600}}{1 - e^{-(280)(2.29)/3{,}600}} = 1{,}238 \text{ veh/h}$$

Similarly, the potential capacity for the minor-street right-turn movement $c_{p,9}$ is computed as follows:

$$c_{p,9} = 260\frac{e^{-(260)(6.3)/3{,}600}}{1 - e^{-(260)(3.39)/3{,}600}} = 760 \text{ veh/h}$$

Finally, the potential capacity for the minor-street left-turn movement $c_{p,7}$ is

$$c_{p,7} = 880\frac{e^{-(880)(6.5)/3{,}600}}{1 - e^{-(880)(3.59)/3{,}600}} = 308 \text{ veh/h}$$

There are no upstream signals, so the adjustments for upstream signals are ignored.

Step 6: Compute Rank 1 Movement Capacities

There are no pedestrians at the intersection; therefore, all pedestrian impedance factors are equal to 1.0, and this step can be ignored.

Step 7: Compute Rank 2 Movement Capacities

The movement capacity for the major-street left-turn movement (Rank 2) $c_{m,4}$ is computed as follows from Equation 20-36:

$$c_{m,4} = c_{p,4} = 1{,}238 \text{ veh/h}$$

Similarly, the movement capacity for the minor-street right-turn movement (Rank 2) $c_{m,9}$ is computed with Equation 20-37:

$$c_{m,9} = c_{p,9} = 760 \text{ veh/h}$$

Step 8: Compute Rank 3 Movement Capacities

The computation of vehicle impedance effects accounts for the reduction in potential capacity due to the impacts of the congestion of a high-priority movement on lower-priority movements.

Major-street movements of Rank 1 and Rank 2 are assumed to be unimpeded by other vehicular movements. Minor-street movements of Rank 3 can be impeded by major-street left-turn movements due to a major-street left-turning vehicle waiting for an acceptable gap at the same time as vehicles of Rank 3. The magnitude of this impedance depends on the probability that major-street left-turning vehicles will be waiting for an acceptable gap at the same time as vehicles of Rank 3. In this example, only the minor-street left-turn movement is defined as a Rank 3 movement. Therefore, the probability of the major-street left-turn movement operating in a queue-free state ($p_{0,4}$) is computed from Equation 20-42:

$$p_{0,4} = 1 - \frac{v_4}{c_{m,4}} = 1 - \frac{160}{1{,}238} = 0.871$$

The movement capacity for the minor-street left-turn movement (Rank 3) $c_{m,7}$ is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor for the minor-street left-turn movement f_7 is computed with Equation 20-46:

$$f_7 = \prod_j p_{0,j} = 0.871$$

The movement capacity for the minor-street left-turn movement (Rank 3) $c_{m,7}$ is computed with Equation 20-47:

$$c_{m,7} = c_{p,7} \times f_7 = 308(0.871) = 268 \text{ veh/h}$$

Step 9: Compute Rank 4 Movement Capacities

There are no Rank 4 movements in this example problem, so this step does not apply.

Step 10: Compute Capacity Adjustment Factors

In this example, the minor-street approach is a single lane shared by right-turn and left-turn movements; therefore, the capacity of these two movements must be adjusted to compute an approach capacity based on shared-lane effects.

The shared-lane capacity for the northbound minor-street approach $c_{SH,NB}$ is computed from Equation 20-59:

$$c_{SH,NB} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}} = \frac{v_7 + v_9}{\frac{v_7}{c_{m,7}} + \frac{v_9}{c_{m,9}}} = \frac{40 + 120}{\frac{40}{268} + \frac{120}{760}} = 521 \text{ veh/h}$$

No other adjustments apply.

Step 11: Compute Control Delay

The control delay computation for any movement includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The control delay for the major-street left-turn movement (Rank 2) d_4 is computed with Equation 20-64:

$$d = \frac{3{,}600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5$$

$$d_4 = \frac{3{,}600}{1{,}238} + 900(0.25)\left[\frac{160}{1{,}238} - 1 + \sqrt{\left(\frac{160}{1{,}238} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}238}\right)\left(\frac{160}{1{,}238}\right)}{450(0.25)}}\right] + 5$$

$$d_4 = 8.3 \text{ s}$$

On the basis of Exhibit 20-2, the westbound left-turn movement is assigned level of service (LOS) A.

The control delay for the minor-street right-turn and left-turn movements is computed by using the same formula; however, one significant difference from the major-street left-turn computation of control delay is that these movements share the same lane. Therefore, the control delay is computed for the approach as a whole, and the shared-lane volume and shared-lane capacity must be used as follows:

$$d_{SH,NB} = \frac{3{,}600}{521} + 900(0.25)\left[\frac{160}{521} - 1 + \sqrt{\left(\frac{160}{521} - 1\right)^2 + \frac{\left(\frac{3{,}600}{521}\right)\left(\frac{160}{521}\right)}{450(0.25)}}\right] + 5$$

$$d_{SH,NB} = 14.9 \text{ s}$$

On the basis of Exhibit 20-2, the northbound approach is assigned LOS B.

Step 11b: Compute Control Delay to Rank 1 Movements

This step is not applicable as the westbound major-street through movement v_5 and westbound major-street left-turn movement v_4 have exclusive lanes at this intersection. It is assumed the eastbound through movement v_2 and eastbound major-street right-turn movement v_3 do not incur any delay at this intersection.

Step 12: Compute Approach and Intersection Control Delay

The control delays to all vehicles on the eastbound approach are assumed to be negligible as described in Step 11b. The control delay for the westbound approach $d_{A,WB}$ is computed with Equation 20-66:

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

$$d_{A,WB} = \frac{0(0) + 0(300) + 8.3(160)}{0 + 300 + 160} = 2.9 \text{ s}$$

It is assumed the westbound through movement incurs no control delay at this intersection. The control delay for the northbound approach was computed in Step 11a as $d_{SH,NB}$.

The intersection control delay d_I is computed from Equation 20-67:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB}}{v_{A,EB} + v_{A,WB} + v_{A,NB}}$$

$$d_I = \frac{0(280) + 2.9(460) + 14.9(160)}{280 + 460 + 160} = 4.1 \text{ s}$$

As noted in Chapter 20, neither major-street approach LOS nor intersection LOS is defined.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for the major-street westbound left-turn movement $Q_{95,4}$ is computed from Equation 20-68:

$$Q_{95,4} \approx 900T \left[\frac{v_4}{c_{m,4}} - 1 + \sqrt{\left(\frac{v_4}{c_{m,4}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,4}}\right)\left(\frac{v_x}{c_{m,4}}\right)}{150T}} \right] \left(\frac{c_{m,4}}{3{,}600}\right)$$

$$Q_{95,4} \approx 900(0.25) \left[\frac{160}{1{,}238} - 1 + \sqrt{\left(\frac{160}{1{,}238} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}238}\right)\left(\frac{160}{1{,}238}\right)}{150(0.25)}} \right] \left(\frac{1{,}238}{3{,}600}\right)$$

$$Q_{95,4} = 0.4 \text{ veh}$$

The result of 0.4 vehicles for the 95th percentile queue indicates a queue of more than one vehicle will occur very infrequently for the major-street left-turn movement.

The 95th percentile queue length for the northbound approach is computed by using the same formula. Similar to the control delay computation, the shared-lane volume and shared-lane capacity must be used as shown:

$$Q_{95,NB} \approx 900(0.25) \left[\frac{160}{521} - 1 + \sqrt{\left(\frac{160}{521} - 1\right)^2 + \frac{\left(\frac{3{,}600}{521}\right)\left(\frac{160}{521}\right)}{150(0.25)}} \right] \left(\frac{521}{3{,}600}\right)$$

$$Q_{95,NB} = 1.3 \text{ veh}$$

The result suggests that a queue of more than one vehicle will occur only occasionally for the northbound approach.

Discussion

Overall, the results indicate this three-leg TWSC intersection will operate well with brief delays and little queuing for all minor movements.

TWSC EXAMPLE PROBLEM 2: PEDESTRIAN CROSSING AT A TWSC INTERSECTION

Calculate the pedestrian LOS of a pedestrian crossing of a major street at a TWSC intersection under the following circumstances:

- Scenario A: unmarked crosswalk, no median refuge island;
- Scenario B: marked crosswalk, median refuge island; and
- Scenario C: marked crosswalk, median refuge island, rectangular rapid-flashing beacons (RRFBs).

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Four-lane major street;
- 1,700 peak hour vehicles, bidirectional;
- *K*-factor = 0.08;
- Crosswalk length without median = 46 ft;
- Crosswalk length with median = 20 ft each side of median;
- Observed pedestrian walking speed = 4.0 ft/s;
- Observed pedestrian start-up and end clearance time = 1.0 s; and
- No pedestrian platooning.

Comments

In addition to the input data listed above, information is required on motor vehicle yield rates under the various scenarios. On the basis of an engineering study of similar intersections in the vicinity, it is determined that average motor vehicle yield rates are 0% with unmarked crosswalks, 50% with marked crosswalks and median islands, and 80% with marked crosswalks, median islands, and RRFBs.

Step 1: Identify Two-Stage Crossings

Scenario A does not have two-stage pedestrian crossings, as no median refuge is available. Analysis for Scenarios B and C should assume two-stage crossings. Thus, analysis for Scenarios B and C will combine two pedestrian crossings of 20 ft each to determine the total delay.

Step 2: Determine Critical Headway

Because there is no pedestrian platooning, the critical headway t_c is determined with Equation 20-77:

$$t_c = \frac{L}{S_p} + t_s$$

Scenario A: t_c = (46 ft)/(4.0 ft/s) + 1.0 s = 12.5 s

Scenario B: t_c = (20 ft)/(4.0 ft/s) + 1.0 s = 6.0 s

Scenario C: t_c = (20 ft)/(4.0 ft/s) + 1.0 s = 6.0 s

Step 3: Estimate Probability of a Delayed Crossing

Equation 20-81 and Equation 20-82 are used to calculate P_b, the probability of a blocked lane, and P_d, the probability of a delayed crossing, respectively. In the case of Scenario A, the crossing consists of four lanes. Scenarios B and C have only two lanes, given the two-stage crossing opportunity.

For the single-stage crossing, v is (1,700 veh/h)/(3,600 s/h) = 0.472 veh/s.

For the two-stage crossing, without any information on directional flows, one-half the volume is used, and v is therefore (850 veh/h)/(3,600 s/h) = 0.236 veh/s.

Scenario A:

$$P_b = 1 - e^{\frac{-t_{c,G}v}{N_L}}$$
$$P_d = 1 - (1 - P_b)^{N_L}$$

$$P_b = 1 - e^{\frac{-12.5(0.472)}{4}} = 0.771$$
$$P_d = 1 - (1 - 0.771)^4 = 0.997$$

Scenarios B and C:

$$P_b = 1 - e^{\frac{-6.0(0.236)}{2}} = 0.508$$
$$P_d = 1 - (1 - 0.508)^2 = 0.758$$

Step 4: Calculate Average Delay to Wait for Adequate Gap

Average gap delay d_g and average gap delay when delay is nonzero d_{gd} are calculated by Equation 20-83 and Equation 20-84, respectively.

Scenario A:

$$d_g = \frac{1}{v}\left(e^{vt_{c,G}} - vt_{c,G} - 1\right)$$

$$d_g = \frac{1}{0.472}\left(e^{0.472(12.5)} - 0.472(12.5) - 1\right) = 761 \text{ s}$$

$$d_{gd} = \frac{d_g}{P_d} = \frac{761}{0.997} = 763 \text{ s}$$

Scenarios B and C:

$$d_g = \frac{1}{0.236}\left(e^{0.236(6.0)} - 0.236(6.0) - 1\right) = 7.2 \text{ s}$$

$$d_{gd} = \frac{7.2}{0.758} = 9.5 \text{ s}$$

Step 5: Calculate Average Pedestrian Delay for the Crossing Stage

Under Scenario A, the motorist yielding rate is 0%. Therefore, there is no reduction in delay due to yielding vehicles, and average delay is the same as that shown in Step 4.

$$d_{p,1} = d_{gd} = 761 \text{ s}$$

Under Scenario B, the motorist yielding rate is 50% and the reduced delay due to yielding vehicles is determined using the process described in Step 5. To start, the average headway of those headways less than the group critical headway h is determined using Equation 20-86.

$$h = \frac{1/v - \left(t_{c,G} + 1/v\right) \exp\left[-v\, t_{c,G}\right]}{1 - \exp\left[-v\, t_{c,G}\right]}$$

$$h = \frac{1/0.236 - (6.0 + 1/0.236) \exp[-(0.236)(6.0)]}{1 - \exp[-(0.236)(6.0)]}$$

$$h = 2.3 \text{ s}$$

The average number of potential yielding events before an adequate gap is available n is then

$$n = \text{int}\left(\frac{d_{gd}}{h}\right) = \text{int}\left(\frac{9.5}{2.3}\right) = 4$$

The two-lane crossings require the use of Equation 20-90 to determine $P(Y_i)$.

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{\left(2P_b[1 - P_b]M_y\right) + (P_b^2 M_y^2)}{P_d}\right]$$

$$P(Y_0) = 0$$

$$P(Y_1) = [0.758 - 0]\left[\frac{(2[0.508][1 - 0.508][0.50]) + (0.508^2 0.50^2)}{0.758}\right] = 0.314$$

$$P(Y_2) = [0.758 - 0.314]\left[\frac{(2[0.508][1 - 0.508][0.50]) + (0.508^2 0.50^2)}{0.758}\right] = 0.184$$

$$P(Y_3) = [0.758 - 0.498]\left[\frac{(2[0.508][1 - 0.508][0.50]) + (0.508^2 0.50^2)}{0.758}\right] = 0.108$$

$$P(Y_4) = [0.758 - 0.606]\left[\frac{(2[0.61][1 - 0.61][0.5]) + (0.61^2 0.50^2)}{0.85}\right] = 0.063$$

The results of Equation 20-90 are substituted into Equation 20-85 to determine average pedestrian delay for the first crossing stage.

$$d_{p,1} = \sum_{i=0}^{n} h(i - 0.5)P(Y_i) + \left(P_d - \sum_{i=0}^{n} P(Y_i)\right) d_{gd}$$

$$\begin{aligned} d_{p,1} = \{&[(2.3)(0 - 0.5)(0)] + [(2.3)(1 - 0.5)(0.314)] + [(2.3)(2 - 0.5)(0.184)] \\ &+ [(2.3)(3 - 0.5)(0.108)] + [(2.3)(4 - 0.5)(0.063)]\} \\ &+ \{(0.758 - [0 + 0.314 + 0.184 + 0.108 + 0.063])(9.5)\} \end{aligned}$$

$$d_{p,1} = 2.12 + 0.85 = 3.0 \text{ s}$$

The second stage of the crossing has the same characteristics as the first stage (same conflicting flow rate and same length). Therefore, the average delay for the second stage is the same as for the first stage:

$$d_{p,2} = d_{p,1} = 3.0 \text{ s}$$

Under Scenario C, the motorist yielding rate is 80%. Compared to Scenario B, the different yielding rate only affects the calculation of $P(Y_i)$.

$$P(Y_0) = 0$$

$$P(Y_1) = [0.758 - 0]\left[\frac{(2[0.508][1-0.508][0.80]) + (0.508^2 0.80^2)}{0.758}\right] = 0.565$$

$$P(Y_2) = [0.758 - 0.565]\left[\frac{(2[0.508][1-0.508][0.80]) + (0.508^2 0.80^2)}{0.758}\right] = 0.144$$

$$P(Y_3) = [0.758 - 0.709]\left[\frac{(2[0.508][1-0.508][0.80]) + (0.508^2 0.80^2)}{0.758}\right] = 0.037$$

$$P(Y_4) = [0.758 - 0.746]\left[\frac{(2[0.508][1-0.508][0.80]) + (0.508^2 0.80^2)}{0.758}\right] = 0.009$$

The average pedestrian delay for the first crossing stage is then

$$\begin{aligned} d_{p,1} = \{&[(2.3)(0-0.5)(0)] + [(2.3)(1-0.5)(0.565)] + [(2.3)(2-0.5)(0.144)] \\ &+ [(2.3)(3-0.5)(0.037)] + [(2.3)(4-0.5)(0.009)]\} \\ &+ \{(0.758 - [0 + 0.565 + 0.144 + 0.037 + 0.009])(9.5)\} \end{aligned}$$

$$d_{p,1} = 1.4 + 0.0 = 1.5 \text{ s}$$

The second stage of the crossing has the same characteristics as the first stage (same conflicting flow rate and same length). Therefore, the average delay for the second stage is the same as for the first stage:

$$d_{p,2} = d_{p,1} = 1.5 \text{ s}$$

Step 6: Calculate Average Pedestrian Delay

The average pedestrian delay for the entire crossing is the sum of the delays for the individual crossing stages.

Scenario A = 761 s

Scenario B = 3.0 + 3.0 s = 6.0 s

Scenario C = 1.5 + 1.5 s = 3.0 s

Step 7: Calculate Pedestrian Satisfaction Probabilities and Determine LOS

Under Scenario A, the odds that pedestrians would be satisfied with their crossing experience, relative to being dissatisfied, are determined from Equation 20-96. In this scenario, there are no pedestrian safety countermeasures at the crossing; therefore, the indicator variables I_{RRFB}, I_{MC}, and I_{MR} are all zero. The AADT of the crossing is the peak hour volume divided by the K-factor = 1,700 / 0.08 = 21,250.

In the situation where an arriving pedestrian can cross immediately (i.e., an adequate gap exists or all blocking vehicles yield), $I_{NY} = 0$. The satisfaction odds are then:

$$O(S/D) = \exp(0.9951 - 0.0438V_{KAADT} + 1.9572I_{RRFB} + 0.9843I_{MC} + 1.5496I_{MR} - 1.9059I_{NY})$$

$$O(S/D) = \exp(0.9951 - 0.0438 \times 21.25 + 1.9572 \times 0 + 0.9843 \times 0 + 1.5496 \times 0 - 1.9059 \times 0)$$

$$O(S/D) = 1.066$$

The probabilities of being satisfied and dissatisfied when an arriving pedestrian can cross immediately re then given by Equation 20-97 and Equation 20-198.

$$P(S, \text{no delay}) = \frac{O(S/D)}{O(S/D) + 1} = \frac{1.066}{1.066 + 1} = 51.6\%$$

$$P(D, \text{no delay}) = 1 - P(S, \text{no delay}) = 1 - 0.516 = 48.4\%$$

In the situation where an arriving pedestrian is delayed crossing the street, $I_{NY} = 1$. The resulting odds and probabilities are then

$$O(S/D) = 0.159$$

$$P(S, \text{delay}) = 13.7\%$$

$$P(D, \text{delay}) = 86.3\%$$

The probability of a non-delayed crossing is given by Equation 20-99. The value of P_d was determined in Step 3. The value of $P(Y_1)$ for a four-lane crossing is determined by Equation 20-93; with a 0% yielding rate, this equation results in $P(Y_1) = 0$.

$$P_{nd} = (1 - P_d) + P_d P(Y_1) = (1 - 0.997) + 0.997 \times 0 = 0.003$$

The average proportion of dissatisfied pedestrians is then determined from Equation 20-102.

$$P_D = P_{nd}P(D, \text{no delay}) + (1 - P_{nd})P(D, \text{delay}) = (0.003)(0.484) + (0.997)(0.863) = 0.862$$

From Exhibit 20-3, when half or more of pedestrians would be dissatisfied, the LOS for the crossing is F.

The calculations for Scenario B are similar to Scenario A, except that the indicator variables I_{MC} and I_{MR} now have values of 1 because a marked crosswalk and a median refuge island, respectively, are present. The value of V_{KAADT} remains the same even though the crossing is now performed in two stages. The calculation for Scenario C is similar to Scenario B, except that the indicator variable I_{RFFB} is also 1 because RFFBs are provided. Exhibit 32-7 provides the calculation results for Scenarios B and C.

Exhibit 32-7
TWSC Example Problem 2: Pedestrian Satisfaction Results for Scenarios B and C

Variable	Scenario B	Scenario C
O (S/D, no delay)	13.44	95.15
P (S, no delay)	93.1%	99.0%
P (D, no delay)	6.9%	1.0%
O (S/D, delay)	2.00	14.15
P (S, delay)	66.6%	93.4%
P (D, delay)	33.4%	6.6%
P_d	0.758	0.758
$P(Y_1)$	0.314	0.565
P_{nd}	0.481	0.670
P (D)	0.207	0.029
LOS	C	A

Discussion

Providing a marked crosswalk and a median refuge island improves the LOS from F to C in Scenario B, and the further addition of RRFBs improves the LOS to A in Scenario C.

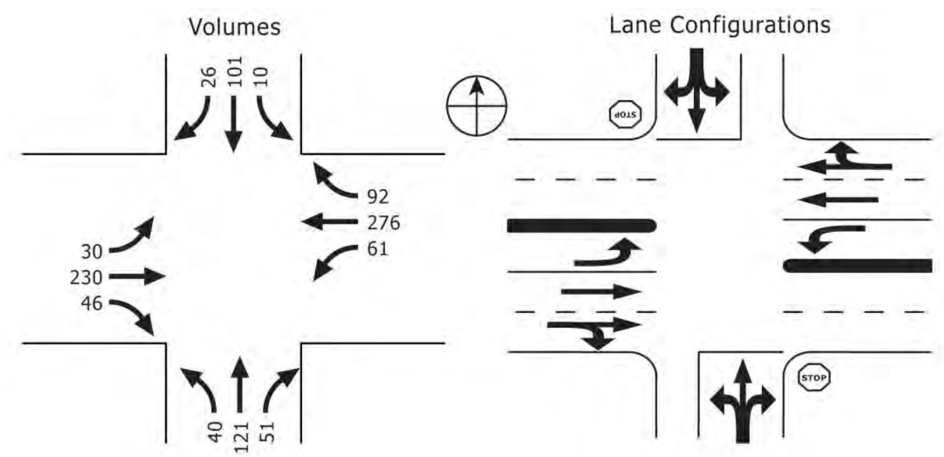
TWSC EXAMPLE PROBLEM 3: FLARED APPROACHES AND MEDIAN STORAGE

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Major street with two lanes in each direction, minor street with one lane on each approach that flares with storage for one vehicle in the flare area, and median storage for two vehicles at one time available for minor-street through and left-turn movements;
- Level grade on all approaches;
- Percentage heavy vehicles on all approaches = 10%;
- Peak hour factor on all approaches = 0.92;
- Length of analysis period = 0.25 h; and
- Volumes and lane configurations as shown in Exhibit 32-8.

Exhibit 32-8
TWSC Example Problem 3: 15-min Volumes and Lane Configurations



Comments

All relevant input parameters are known, so no default values are needed or used.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Because hourly volumes and a peak hour factor have been provided, each hourly volume is divided by the peak hour factor to determine a peak 15-min flow rate (in vehicles per hour) for each movement. These values are shown in Exhibit 32-9.

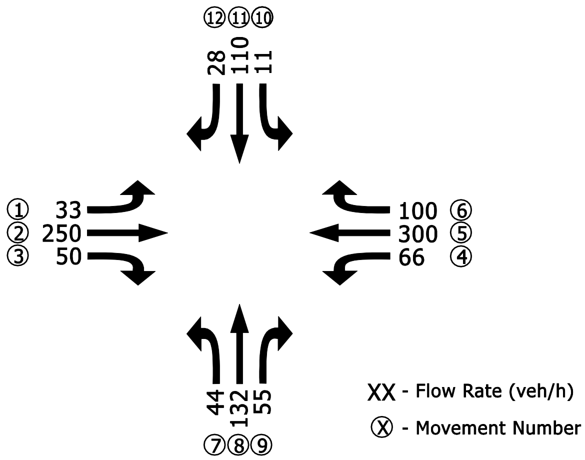


Exhibit 32-9
TWSC Example Problem 3: Movement Numbers and Calculation of Peak 15-min Flow Rates

Step 3: Compute Conflicting Flow Rates

The conflicting flow rates for each minor movement at the intersection are computed according to the equations in Chapter 20. The conflicting flow for the eastbound major-street left-turn movement $v_{c,1}$ is computed according to Equation 20-2 as follows:

$$v_{c,1} = v_5 + v_6 + v_{16} = 300 + 100 + 0 = 400 \text{ veh/h}$$

Similarly, the conflicting flow for the westbound major-street left-turn movement $v_{c,4}$ is computed according to Equation 20-3 as follows:

$$v_{c,4} = v_2 + v_3 + v_{15} = 250 + 50 + 0 = 300 \text{ veh/h}$$

The conflicting flows for the northbound minor-street right-turn movement $v_{c,9}$ and southbound minor-street right-turn movement $v_{c,12}$ are computed with Equation 20-6 and Equation 20-7, respectively, as follows (with no U-turns and pedestrians, the last three terms can be assigned zero):

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15}$$

$$v_{c,9} = 0.5(250) + 0.5(50) + 0 + 0 + 0 = 150 \text{ veh/h}$$

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{1U} + v_{13} + v_{16}$$

$$v_{c,12} = 0.5(300) + 0.5(100) + 0 + 0 + 0 = 200 \text{ veh/h}$$

Next, the conflicting flow for the northbound minor-street through movement $v_{c,8}$ is computed. Because two-stage gap acceptance is available for this movement, the conflicting flow rates shown in Stage I and Stage II must be computed separately. The conflicting flow for Stage I $v_{c,\text{I},8}$ is computed from Equation 20-14:

$$v_{c,\text{I},8} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

$$v_{c,\text{I},8} = 2(33 + 0) + 250 + 0.5(50) + 0 = 341 \text{ veh/h}$$

The conflicting flow for Stage II $v_{c,\text{II},8}$ is computed from Equation 20-16:

$$v_{c,\text{II},8} = 2(v_4 + v_{4U}) + v_5 + v_6 + v_{16}$$

$$v_{c,\text{II},8} = 2(66 + 0) + 300 + 100 + 0 = 532 \text{ veh/h}$$

The total conflicting flow for the northbound through movement $v_{c,8}$ is computed as follows:

$$v_{c,8} = v_{c,\mathrm{I},8} + v_{c,\mathrm{II},8} = 341 + 532 = 873 \text{ veh/h}$$

Similarly, the conflicting flow for the southbound minor-street through movement $v_{c,11}$ is computed in two stages as follows:

$$v_{c,\mathrm{I},11} = 2(66 + 0) + 300 + 0.5(100) + 0 = 482 \text{ veh/h}$$

$$v_{c,\mathrm{II},11} = 2(33 + 0) + 250 + 50 + 0 = 366 \text{ veh/h}$$

$$v_{c,11} = v_{c,\mathrm{I},11} + v_{c,\mathrm{II},11} = 482 + 366 = 848 \text{ veh/h}$$

Next, the conflicting flow for the northbound minor-street left-turn movement $v_{c,7}$ is computed. Because two-stage gap acceptance is available for this movement, the conflicting flow rates shown in Stage I and Stage II must be computed separately. The conflicting flow for Stage I $v_{c,\mathrm{I},7}$ is computed with Equation 20-20 as follows:

$$v_{c,\mathrm{I},7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

$$v_{c,\mathrm{I},7} = 2(33 + 0) + 250 + 0.5(50) + 0 = 341 \text{ veh/h}$$

The conflicting flow for Stage II $v_{c,\mathrm{II},7}$ is computed with Equation 20-26 as follows:

$$v_{c,\mathrm{II},7} = 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$

$$v_{c,\mathrm{II},7} = 2(66 + 0) + 0.5(300) + 0.5(110) + 0 = 337 \text{ veh/h}$$

The total conflicting flow for the northbound left-turn movement $v_{c,7}$ is computed as follows:

$$v_{c,7} = v_{c,\mathrm{I},7} + v_{c,\mathrm{II},7} = 341 + 337 = 678 \text{ veh/h}$$

Similarly, the conflicting flow for the southbound minor-street left-turn movement $v_{c,10}$ is computed in two stages as follows:

$$v_{c,\mathrm{I},10} = 2(66 + 0) + 300 + 0.5(100) + 0 = 482 \text{ veh/h}$$

$$v_{c,\mathrm{II},10} = 2(33 + 0) + 0.5(250) + 0.5(132) + 0 = 257 \text{ veh/h}$$

$$v_{c,10} = v_{c,\mathrm{I},10} + v_{c,\mathrm{II},10} = 482 + 257 = 739 \text{ veh/h}$$

Step 4: Determine Critical Headways and Follow-Up Headways

The critical headway for each minor movement is computed beginning with the base critical headway given in Exhibit 20-12. The base critical headway for each movement is then adjusted according to Equation 20-30. The critical headways for the eastbound and westbound major-street left turns $t_{c,1}$ and $t_{c,4}$ (in this case, $t_{c,1} = t_{c,4}$) are computed as follows:

$$t_{c,1} = t_{c,4} = t_{c,\text{base}} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,1} = t_{c,4} = 4.1 + 2.0(0.1) + 0(0) - 0 = 4.3 \text{ s}$$

Next, the critical headways for the northbound and southbound minor-street right-turn movements $t_{c,9}$ and $t_{c,12}$ (in this case, $t_{c,9} = t_{c,12}$) are computed as follows:

$$t_{c,9} = t_{c,12} = 6.9 + 2.0(0.1) + 0.1(0) - 0 = 7.1 \text{ s}$$

Next, the critical headways for the northbound and southbound minor-street through movements $t_{c,8}$ and $t_{c,11}$ (in this case, $t_{c,8} = t_{c,11}$) are computed. Because two-stage gap acceptance is available for these movements, the critical headways for Stage I and Stage II must be computed, along with the critical headways for these movements assuming single-stage gap acceptance. The critical headways for Stage I and Stage II, $t_{c,\mathrm{I},8}$, $t_{c,I,11}$ and $t_{c,\mathrm{II},8}$, $t_{c,\mathrm{II},11}$, respectively (in this case, $t_{c,\mathrm{I},8} = t_{c,\mathrm{II},8} = t_{c,\mathrm{I},11} = t_{c,\mathrm{II},11}$), are computed as follows:

$$t_{c,\mathrm{I},8} = t_{c,\mathrm{II},8} = t_{c,\mathrm{I},11} = t_{c,\mathrm{II},11} = 5.5 + 2.0(0.1) + 0.2(0) - 0 = 5.7 \text{ s}$$

The critical headways for $t_{c,8}$ and $t_{c,11}$ (in this case, $t_{c,8} = t_{c,11}$), assuming single-stage gap acceptance, are computed as follows:

$$t_{c,8} = t_{c,11} = 6.5 + 2.0(0.1) + 0.2(0) - 0 = 6.7 \text{ s}$$

Finally, the critical headways for the northbound and southbound minor-street left-turn movements $t_{c,7}$ and $t_{c,10}$ (in this case, $t_{c,7} = t_{c,10}$) are computed. Because two-stage gap acceptance is available for these movements, the critical headways for Stage I and Stage II must be computed, along with the critical headways for these movements assuming single-stage gap acceptance. The critical headways for Stage I and Stage II, $t_{c,\mathrm{I},7}$, $t_{c,\mathrm{I},10}$ and $t_{c,\mathrm{II},7}$, $t_{c,\mathrm{II},10}$, respectively (in this case, $t_{c,\mathrm{I},7} = t_{c,\mathrm{II},7} = t_{c,\mathrm{I},10} = t_{c,\mathrm{II},10}$), are computed as follows:

$$t_{c,\mathrm{I},7} = t_{c,\mathrm{II},7} = t_{c,\mathrm{I},10} = t_{c,\mathrm{II},10} = 6.5 + 2.0(0.1) + 0.2(0) - 0 = 6.7 \text{ s}$$

The critical headways for $t_{c,7}$ and $t_{c,10}$ (in this case, $t_{c,7} = t_{c,10}$), assuming single-stage gap acceptance, are computed as follows:

$$t_{c,7} = t_{c,10} = 7.5 + 2.0(0.1) + 0.2(0) - 0 = 7.7 \text{ s}$$

The follow-up headway for each minor movement is computed beginning with the base follow-up headway given in Exhibit 20-13. The base follow-up headway for each movement is then adjusted according to Equation 20-31. The follow-up headways for the northbound and southbound major-street left-turn movements $t_{f,1}$ and $t_{f,4}$ (in this case, $t_{f,1} = t_{f,4}$) are computed as follows:

$$t_{f,1} = t_{f,4} = t_{f,base} + t_{f,HV} P_{HV}$$

$$t_{f,1} = t_{f,4} = 2.2 + 1.0(0.1) = 2.3 \text{ s}$$

Next, the follow-up headways for the northbound and southbound minor-street right-turn movements $t_{f,9}$ and $t_{f,12}$ (in this case, $t_{f,9} = t_{f,12}$) are computed as follows:

$$t_{f,9} = t_{f,12} = 3.3 + 1.0(0.1) = 3.4 \text{ s}$$

Next, the follow-up headways for the northbound and southbound minor-street through movements $t_{f,8}$ and $t_{f,11}$ (in this case, $t_{f,8} = t_{f,11}$) are computed as follows:

Follow-up headways for the minor-street through and left-turn movements are computed for the movement as a whole. Follow-up headways are not broken up by stage because they apply only to vehicles as they exit the approach and enter the intersection.

$$t_{f,8} = t_{f,11} = 4.0 + 1.0(0.1) = 4.1 \text{ s}$$

Finally, the follow-up headways for the northbound and southbound minor-street left-turn movements $t_{f,7}$ and $t_{f,10}$ (in this case, $t_{f,7} = t_{f,10}$) are computed as follows:

$$t_{f,7} = t_{f,10} = 3.5 + 1.0(0.1) = 3.6 \text{ s}$$

Step 5: Compute Potential Capacities

Because no upstream signals are present, the procedure in Step 5a is followed.

The computation of a potential capacity for each movement provides the analyst with a definition of capacity under the assumed base conditions. The potential capacity will be adjusted in later steps to estimate the movement capacity for each movement. The potential capacity for each movement is a function of the conflicting flow rate, critical headway, and follow-up headway computed in the previous steps. The potential capacity for the northbound major-street left-turn movement $c_{p,1}$ is computed from Equation 20-32:

$$c_{p,1} = v_{c,1} \frac{e^{-v_{c,1} t_{c,1}/3{,}600}}{1 - e^{-v_{c,1} t_{f,1}/3{,}600}}$$

$$c_{p,1} = 400 \frac{e^{-(400)(4.3)/3{,}600}}{1 - e^{-(400)(2.3)/3{,}600}} = 1{,}100 \text{ veh/h}$$

Similarly, the potential capacities for Movements 4, 9, and 12 ($c_{p,4}$, $c_{p,9}$, and $c_{p,12}$, respectively) are computed as follows:

$$c_{p,4} = 300 \frac{e^{-(300)(4.3)/3{,}600}}{1 - e^{-(300)(2.3)/3{,}600}} = 1{,}202 \text{ veh/h}$$

$$c_{p,9} = 150 \frac{e^{-(150)(7.1)/3{,}600}}{1 - e^{-(150)(3.4)/3{,}600}} = 845 \text{ veh/h}$$

$$c_{p,12} = 200 \frac{e^{-(200)(7.1)/3{,}600}}{1 - e^{-(200)(3.4)/3{,}600}} = 783 \text{ veh/h}$$

Because the two-stage gap-acceptance adjustment procedure will be implemented for estimating the capacity of the minor-street movements, three potential capacity values must be computed for each of Movements 7, 8, 10, and 11. First, the potential capacity must be computed for Stage I, $c_{p,I,8}$, $c_{p,I,11}$, $c_{p,I,7}$, and $c_{p,I,10}$, for each movement as follows:

$$c_{p,I,8} = 341 \frac{e^{-(341)(5.7)/3{,}600}}{1 - e^{-(341)(4.1)/3{,}600}} = 618 \text{ veh/h}$$

$$c_{p,I,11} = 482 \frac{e^{-(482)(5.7)/3{,}600}}{1 - e^{-(482)(4.1)/3{,}600}} = 532 \text{ veh/h}$$

$$c_{p,I,7} = 341 \frac{e^{-(341)(6.7)/3{,}600}}{1 - e^{-(341)(3.6)/3{,}600}} = 626 \text{ veh/h}$$

$$c_{p,I,10} = 482 \frac{e^{-(482)(6.7)/3{,}600}}{1 - e^{-(482)(3.6)/3{,}600}} = 514 \text{ veh/h}$$

Next, the potential capacity must be computed for Stage II for each movement, $c_{p,II,8}$, $c_{p,II,11}$, $c_{p,II,7}$, and $c_{p,II,10}$, as follows:

$$c_{p,II,8} = 532 \frac{e^{-(532)(5.7)/3{,}600}}{1 - e^{-(532)(4.1)/3{,}600}} = 504 \text{ veh/h}$$

$$c_{p,II,11} = 366 \frac{e^{-(366)(5.7)/3{,}600}}{1 - e^{-(366)(4.1)/3{,}600}} = 601 \text{ veh/h}$$

$$c_{p,II,7} = 337 \frac{e^{-(337)(6.7)/3{,}600}}{1 - e^{-(337)(3.6)/3{,}600}} = 629 \text{ veh/h}$$

$$c_{p,I,10} = 257 \frac{e^{-(257)(6.7)/3{,}600}}{1 - e^{-(257)(3.6)/3{,}600}} = 703 \text{ veh/h}$$

Finally, the potential capacity must be computed assuming single-stage gap acceptance for each movement, $c_{p,8}$, $c_{p,11}$, $c_{p,7}$, and $c_{p,10}$, as follows:

$$c_{p,8} = 873 \frac{e^{-(873)(6.7)/3{,}600}}{1 - e^{-(873)(4.1)/3{,}600}} = 273 \text{ veh/h}$$

$$c_{p,11} = 848 \frac{e^{-(848)(6.7)/3{,}600}}{1 - e^{-(848)(4.1)/3{,}600}} = 283 \text{ veh/h}$$

$$c_{p,7} = 678 \frac{e^{-(678)(7.7)/3{,}600}}{1 - e^{-(678)(3.6)/3{,}600}} = 323 \text{ veh/h}$$

$$c_{p,10} = 739 \frac{e^{-(739)(7.7)/3{,}600}}{1 - e^{-(739)(3.6)/3{,}600}} = 291 \text{ veh/h}$$

Steps 6–9: Compute Movement Capacities

Because no pedestrians are present, the procedures given in Chapter 20 are followed.

Step 6: Compute Rank 1 Movement Capacities

There is no computation for this step.

Step 7: Compute Rank 2 Movement Capacities

Step 7a: Movement Capacity for Major-Street Left-Turn Movements

The movement capacity of each Rank 2 major-street left-turn movement is equal to its potential capacity:

$$c_{m,1} = c_{p,1} = 1{,}100 \text{ veh/h}$$

$$c_{m,4} = c_{p,4} = 1{,}202 \text{ veh/h}$$

Step 7b: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity of each minor-street right-turn movement is equal to its potential capacity:

$$c_{m,9} = c_{p,9} = 845 \text{ veh/h}$$

$$c_{m,12} = c_{p,12} = 783 \text{ veh/h}$$

Step 7c: Movement Capacity for Major-Street U-Turn Movements

No U-turns are present, so this step is skipped.

Step 7d: Effect of Major-Street Shared Through and Left-Turn Lane

Separate major-street left-turn lanes are provided, so this step is skipped.

Step 8: Compute Rank 3 Movement Capacities

The movement capacity of each Rank 3 movement is equal to its potential capacity, factored by any impedance due to conflicting pedestrian or vehicular movements.

Step 8a: Rank 3 Capacity for One-Stage Movements

As there are no pedestrians assumed at this intersection, the Rank 3 movements will be impeded only by other vehicular movements. Specifically, the Rank 3 movements will be impeded by major-street left-turning traffic, and as a first step in determining the impact of this impedance, the probability that these movements will operate in a queue-free state must be computed according to Equation 20-42:

$$p_{0,1} = 1 - \frac{v_1}{c_{m,1}} = 1 - \frac{33}{1{,}100} = 0.970$$

$$p_{0,4} = 1 - \frac{66}{1{,}202} = 0.945$$

Next, by using the probabilities computed above, capacity adjustment factors f_8 and f_{11} can be computed according to Equation 20-46:

$$f_8 = f_{11} = p_{0,1} \times p_{0,4} = (0.970)(0.945) = 0.917$$

Finally, under the single-stage gap-acceptance assumption, the movement capacities $c_{m,8}$ and $c_{m,11}$ can be computed according to Equation 20-47:

$$c_{m,8} = c_{p,8} \times f_8 = (273)(0.917) = 250 \text{ veh/h}$$

$$c_{m,11} = c_{p,11} \times f_{11} = (283)(0.917) = 260 \text{ veh/h}$$

Because Movements 8 and 11 will operate under two-stage gap acceptance, the capacity adjustment procedure for estimating the capacity of Stage I and Stage II of these movements must be completed.

To begin the process of estimating Stage I and Stage II movement capacities, the probabilities of queue-free states on conflicting Rank 2 movements calculated above are entered into Equation 20-46 as before, but this time capacity adjustment factors are estimated for each individual stage as follows:

$$f_{\mathrm{I},8} = p_{0,1} = 0.970$$

$$f_{\mathrm{I},11} = p_{0,4} = 0.945$$

$$f_{\mathrm{II},8} = p_{0,4} = 0.945$$

$$f_{\mathrm{II},11} = p_{0,1} = 0.970$$

The Stage I movement capacities are then computed as follows:

$$c_{m,\mathrm{I},8} = c_{p,\mathrm{I},8} \times f_{I,8} = (618)(0.970) = 599 \text{ veh/h}$$

$$c_{m,\mathrm{I},11} = c_{p,\mathrm{I},11} \times f_{\mathrm{I},11} = (532)(0.945) = 503 \text{ veh/h}$$

The Stage II movement capacities are then computed as follows:

$$c_{m,\mathrm{II},8} = c_{p,\mathrm{II},8} \times f_{\mathrm{II},8} = (504)(0.945) = 476 \text{ veh/h}$$

$$c_{m,\mathrm{II},11} = c_{p,\mathrm{II},11} \times f_{\mathrm{II},11} = (601)(0.970) = 583 \text{ veh/h}$$

Step 8b: Rank 3 Capacity for Two-Stage Movements

The two-stage gap-acceptance procedure will result in a total capacity estimate for Movements 8 and 11. To begin the procedure, an adjustment factor a must be computed for each movement by using Equation 20-48, under the assumption there is storage for two vehicles in the median refuge area; thus, $n_m = 2$.

$$a_8 = a_{11} = 1 - 0.32e^{-1.3\sqrt{n_m}} = 1 - 0.32e^{-1.3\sqrt{2}} = 0.949$$

Next, an intermediate variable, y, must be computed for each movement by using Equation 20-49:

$$y_8 = \frac{c_{m,I,8} - c_{m,8}}{c_{m,II,8} - v_1 - c_{m,8}} = \frac{599 - 250}{476 - 33 - 250} = 1.808$$

$$y_{11} = \frac{c_{m,I,11} - c_{m,11}}{c_{m,II,11} - v_4 - c_{m,11}} = \frac{503 - 260}{583 - 66 - 260} = 0.946$$

Finally, the total capacity for each movement $c_{T,8}$ and $c_{T,11}$ is computed according to Equation 20-50, because $y \neq 1$:

$$c_{m,T,8} = \frac{a_8}{y_8^{n_m+1} - 1}\left[y_8\left(y_8^{n_m} - 1\right)\left(c_{m,II,8} - v_1\right) + (y_8 - 1)c_{m,8}\right]$$

$$c_{m,T,8} = \frac{0.949}{1.808^{2+1} - 1}[(1.808)(1.808^2 - 1)(476 - 33) + (1.808 - 1)(250)]$$

$$c_{m,T,8} = 390 \text{ veh/h}$$

$$c_{m,T,11} = \frac{a_{11}}{y_{11}^{n_m+1} - 1}\left[y_{11}\left(y_{11}^{n_m} - 1\right)\left(c_{m,II,11} - v_4\right) + (y_{11} - 1)c_{m,11}\right]$$

$$c_{m,T,11} = \frac{0.949}{0.946^{2+1} - 1}[(0.946)(0.946^2 - 1)(583 - 66) + (0.946 - 1)(260)]$$

$$c_{m,T,11} = 405 \text{ veh/h}$$

Step 9: Compute Rank 4 Movement Capacities

Step 9a: Rank 4 Capacity for One-Stage Movements

The vehicle impedance effects for Rank 4 movements are first estimated by assuming single-stage gap acceptance. Rank 4 movements are impeded by all the same movements impeding Rank 2 and Rank 3 movements with the addition of impedances due to the minor-street crossing movements and minor-street right-turn movements. The probability that these movements will operate in a queue-free state must be incorporated into the procedure.

The probabilities that the minor-street right-turn movements will operate in a queue-free state ($p_{0,9}$ and $p_{0,12}$) are computed as follows:

$$p_{0,9} = 1 - \frac{v_9}{c_{m,9}} = 1 - \frac{55}{845} = 0.935$$

$$p_{0,12} = 1 - \frac{28}{783} = 0.964$$

To compute p', the probability that both the major-street left-turn movements and the minor-street crossing movements will operate in a queue-free state simultaneously, the analyst must first compute $p_{0,k}$, which is done in the same manner as the computation of $p_{0,j}$, except k represents Rank 3 movements. The values for $p_{0,k}$ are computed as follows:

$$p_{0,8} = 1 - \frac{v_8}{c_{m,T,8}} = 1 - \frac{132}{390} = 0.662$$

$$p_{0,11} = 1 - \frac{110}{405} = 0.728$$

Next, the analyst must compute p'', which, under the single-stage gap-acceptance assumption, is simply the product of f_j and $p_{0,k}$. The value for $f_8 = f_{11} = 0.917$ is as computed above. The value for $p_{0,11}$ is computed by using the total capacity for Movement 11 calculated in the previous step:

$$p''_7 = p_{0,11} \times f_{11} = (0.728)(0.917) = 0.668$$

$$p''_{10} = p_{0,8} \times f_8 = (0.662)(0.917) = 0.607$$

With the values for p'', the probability of a simultaneous queue-free state for each movement can be computed by using Equation 20-52 as follows:

$$p'_7 = 0.65p''_7 - \frac{p''_7}{p''_7 + 3} + 0.6\sqrt{p''_7}$$

$$p'_7 = 0.65(0.668) - \frac{0.668}{0.668 + 3} + 0.6\sqrt{0.668} = 0.742$$

$$p'_{10} = 0.65(0.607) - \frac{0.607}{0.607 + 3} + 0.6\sqrt{0.607} = 0.694$$

Next, with the probabilities computed above, capacity adjustment factors f_7 and f_{10} can be computed according to Equation 20-53:

$$f_7 = p'_7 \times p_{0,12} = (0.742)(0.964) = 0.715$$

$$f_{10} = p'_{10} \times p_{0,9} = (0.694)(0.935) = 0.649$$

Finally, under the single-stage gap-acceptance assumption, the movement capacities $c_{m,7}$ and $c_{m,10}$ can be computed according to Equation 20-54:

$$c_{m,7} = c_{p,7} \times f_7 = (323)(0.715) = 231 \text{ veh/h}$$

$$c_{m,10} = c_{p,10} \times f_{10} = (291)(0.649) = 189 \text{ veh/h}$$

Step 9b: Rank 4 Capacity for Two-Stage Movements

Similar to the minor-street crossing movements at this intersection, Movements 7 and 10 will also operate under two-stage gap acceptance. Therefore, the capacity adjustment procedure for estimating the capacity of Stage I and Stage II of these movements must be completed.

Under the assumption of two-stage gap acceptance with a median refuge area, the minor-street left-turn movements operate as Rank 3 movements in each individual stage of completing the left-turn maneuver. To begin the process of estimating two-stage movement capacities, the probabilities of queue-free states on conflicting Rank 2 movements for Stage I of the minor-street left-turn movement are entered into Equation 20-46, and capacity adjustment factors for Stage I are computed as follows:

$$f_{\text{I},7} = p_{0,1} = 0.970$$

$$f_{\text{I},10} = p_{0,4} = 0.945$$

The Stage I movement capacities can then be computed as follows:

$$c_{m,\mathrm{I},7} = c_{p,\mathrm{I},7} \times f_{\mathrm{I},7} = (626)(0.970) = 607 \text{ veh/h}$$

$$c_{m,\mathrm{I},10} = c_{p,\mathrm{I},10} \times f_{\mathrm{I},10} = (514)(0.945) = 486 \text{ veh/h}$$

Next, the probabilities of queue-free states on conflicting Rank 2 movements for Stage II of the minor-street left-turn movement are entered into Equation 20-46. However, before estimating these probabilities, the probability of a queue-free state for the first stage of the minor-street crossing movement must be estimated as it impedes Stage II of the minor-street left-turn movement. These probabilities are estimated with Equation 20-42:

$$p_{0,\mathrm{I},8} = 1 - \frac{v_8}{c_{m,\mathrm{I},8}} = 1 - \frac{132}{599} = 0.780$$

$$p_{0,\mathrm{I},11} = 1 - \frac{110}{503} = 0.781$$

The capacity adjustment factors for Stage II are then computed as follows:

$$f_{\mathrm{II},7} = p_{0,4} \times p_{0,12} \times p_{0,\mathrm{I},11} = (0.945)(0.964)(0.781) = 0.711$$

$$f_{\mathrm{II},10} = p_{0,1} \times p_{0,9} \times p_{0,\mathrm{I},8} = (0.970)(0.935)(0.780) = 0.707$$

Finally, the movement capacities for Stage II are computed as follows:

$$c_{m,\mathrm{II},7} = c_{p,\mathrm{II},7} \times f_{\mathrm{II},7} = (629)(0.711) = 447 \text{ veh/h}$$

$$c_{m,\mathrm{II},10} = (703)(0.707) = 497 \text{ veh/h}$$

The final result of the two-stage gap-acceptance procedure will be a total capacity estimate for Movements 7 and 10. To begin the procedure, an adjustment factor a must be computed for each movement by using Equation 20-55, under the assumption there is storage for two vehicles in the median refuge area; thus, $n_m = 2$.

$$a_7 = a_{10} = 1 - 0.32e^{-1.3\sqrt{n_m}} = 1 - 0.32e^{-1.3\sqrt{2}} = 0.949$$

Next, an intermediate variable y must be computed for each movement by using Equation 20-56:

$$y_7 = \frac{c_{m,\mathrm{I},7} - c_{m,7}}{c_{m,\mathrm{II},7} - v_1 - c_{m,7}} = \frac{607 - 231}{447 - 33 - 231} = 2.055$$

$$y_{10} = \frac{c_{m,\mathrm{I},10} - c_{m,10}}{c_{m,\mathrm{II},10} - v_4 - c_{m,10}} = \frac{486 - 189}{497 - 66 - 189} = 1.227$$

Finally, the total capacity for each movement, $c_{T,7}$ and $c_{T,10}$, is computed according to Equation 20-57, as $y \neq 1$:

$$c_{T,7} = \frac{a_7}{y_7^{n_m+1} - 1}\left[y_7\left(y_7^{n_m} - 1\right)\left(c_{m,II,7} - v_1\right) + (y_7 - 1)c_{m,7}\right]$$

$$c_{T,7} = \frac{0.949}{2.055^{2+1} - 1}[(2.055)(2.055^2 - 1)(447 - 33) + (2.055 - 1)(231)]$$

$$c_{T,7} = 369 \text{ veh/h}$$

$$c_{T,10} = \frac{a_{10}}{y_{10}^{n_m+1} - 1}\left[y_{10}\left(y_{10}^{n_m} - 1\right)\left(c_{m,II,10} - v_4\right) + (y_{10} - 1)c_{m,10}\right]$$

$$c_{T,10} = \frac{0.949}{1.227^{2+1} - 1}[(1.227)(1.227^2 - 1)(497 - 66) + (1.227 - 1)(189)]$$

$$c_{T,10} = 347 \text{ veh/h}$$

Step 10: Compute Final Capacity Adjustments

In this example problem, several final capacity adjustments must be made to account for the effect of the shared lanes and the flared lanes on the minor-street approaches. Initially, the shared-lane capacities for each of the minor-street approaches must be computed on the assumption of no flared lanes; after these computations are completed, the effects of the flare can be incorporated to compute an actual capacity for each minor-street approach.

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

In this example, both minor-street approaches have single-lane entries, meaning that all movements on the minor street share one lane. The shared-lane capacities for the minor-street approaches are computed according to Equation 20-59:

$$c_{SH,NB} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}} = \frac{v_7 + v_8 + v_9}{\frac{v_7}{c_{m,7}} + \frac{v_8}{c_{m,8}} + \frac{v_9}{c_{m,9}}} = \frac{44 + 132 + 55}{\frac{44}{369} + \frac{132}{390} + \frac{55}{845}} = 442 \text{ veh/h}$$

$$c_{SH,SB} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}} = \frac{11 + 110 + 28}{\frac{11}{347} + \frac{110}{405} + \frac{28}{783}} = 439 \text{ veh/h}$$

Step 10b: Flared Minor-Street Lane Effects

In this example, the capacity of each minor-street approach will be greater than the shared capacities computed in the previous step due to the shared-lane condition on each approach. On each approach, it is assumed one vehicle at a time can queue in the flared area; therefore, $n = 1$.

First, the analyst must estimate the average queue length for each movement sharing the lane on each approach. Required input data for this estimation include the flow rates and control delays for each movement. Although the flow rates are known input data, the control delays have not yet been computed. Therefore, the control delay for each movement, assuming a 15-min analysis period and separate lanes for each movement, is computed with Equation 20-64:

$$d_7 = \frac{3{,}600}{c_7} + 900T\left[\frac{v_7}{c_{m,7}} - 1 + \sqrt{\left(\frac{v_7}{c_{m,7}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,7}}\right)\left(\frac{v_7}{c_{m,7}}\right)}{450T}}\right] + 5$$

$$d_7 = \frac{3{,}600}{369} + 900(0.25)\left[\frac{44}{369} - 1 + \sqrt{\left(\frac{44}{369} - 1\right)^2 + \frac{\left(\frac{3{,}600}{369}\right)\left(\frac{44}{369}\right)}{450(0.25)}}\right] + 5$$

$$d_7 = 16.07 \text{ s}$$

$$d_8 = \frac{3{,}600}{390} + 900(0.25)\left[\frac{132}{390} - 1 + \sqrt{\left(\frac{132}{390} - 1\right)^2 + \frac{\left(\frac{3{,}600}{390}\right)\left(\frac{132}{390}\right)}{450(0.25)}}\right] + 5$$

$$d_8 = 18.88 \text{ s}$$

$$d_9 = \frac{3{,}600}{845} + 900(0.25)\left[\frac{55}{845} - 1 + \sqrt{\left(\frac{55}{845} - 1\right)^2 + \frac{\left(\frac{3{,}600}{845}\right)\left(\frac{55}{845}\right)}{450(0.25)}}\right] + 5$$

$$d_9 = 9.57 \text{ s}$$

$$d_{10} = \frac{3{,}600}{347} + 900(0.25)\left[\frac{11}{347} - 1 + \sqrt{\left(\frac{11}{347} - 1\right)^2 + \frac{\left(\frac{3{,}600}{347}\right)\left(\frac{11}{347}\right)}{450(0.25)}}\right] + 5$$

$$d_{10} = 15.71 \text{ s}$$

$$d_{11} = \frac{3{,}600}{405} + 900(0.25)\left[\frac{110}{405} - 1 + \sqrt{\left(\frac{110}{405} - 1\right)^2 + \frac{\left(\frac{3{,}600}{405}\right)\left(\frac{110}{405}\right)}{450(0.25)}}\right] + 5$$

$$d_{11} = 17.17 \text{ s}$$

$$d_{12} = \frac{3{,}600}{783} + 900(0.25)\left[\frac{28}{783} - 1 + \sqrt{\left(\frac{28}{783} - 1\right)^2 + \frac{\left(\frac{3{,}600}{783}\right)\left(\frac{28}{783}\right)}{450(0.25)}}\right] + 5$$

$$d_{12} = 9.77 \text{ s}$$

In this example, all movements on the minor-street approach share one lane; therefore, the average queue lengths for each minor-street movement are computed as follows from Equation 20-60:

$$Q_{sep,7} = \frac{d_{sep,7} v_{sep,7}}{3{,}600} = \frac{(16.07)(44)}{3{,}600} = 0.20 \text{ veh}$$

$$Q_{sep,8} = \frac{(18.88)(132)}{3{,}600} = 0.69 \text{ veh}$$

$$Q_{sep,9} = \frac{(9.57)(55)}{3{,}600} = 0.15 \text{ veh}$$

$$Q_{sep,10} = \frac{(15.71)(11)}{3{,}600} = 0.05 \text{ veh}$$

$$Q_{sep,11} = \frac{(17.17)(110)}{3{,}600} = 0.53 \text{ veh}$$

$$Q_{sep,12} = \frac{(9.77)(28)}{3{,}600} = 0.08 \text{ veh}$$

Next, the required length of the storage area so that each approach would operate effectively as separate lanes is computed with Equation 20-61:

$$n_{Max} = \max_i\left[\text{round}\left(Q_{sep,i} + 1\right)\right]$$

$$n_{Max,NB} = \max_{NB}[\text{round}(Q_{sep,7}+1),\text{round}(Q_{sep,8}+1),\text{round}(Q_{sep,9}+1)]$$

$$n_{Max,NB} = \max_{NB}[\text{round}(0.20+1),\text{round}(0.69+1),\text{round}(0.15+1)] = 2$$

$$n_{Max,SB} = \max_{SB}[\text{round}(0.05+1),\text{round}(0.53+1),\text{round}(0.08+1)] = 2$$

The next step involves estimating separate lane capacities, with consideration of the limitation of the amount of right-turn traffic that could actually move into a separate right-turn lane given a queue before the location of the flare. To compute separate lane capacities, the shared-lane capacities of the through plus left-turn movement on each approach must first be estimated according to Equation 20-59:

$$c_{L+TH,NB} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}} = \frac{v_7+v_8}{\frac{v_7}{c_{m,7}}+\frac{v_8}{c_{m,8}}} = \frac{44+132}{\frac{44}{369}+\frac{132}{390}} = 385 \text{ veh/h}$$

$$c_{L+TH,SB} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}} = \frac{v_{10}+v_{11}}{\frac{v_{10}}{c_{m,10}}+\frac{v_{11}}{c_{m,11}}} = \frac{11+110}{\frac{11}{347}+\frac{110}{405}} = 399 \text{ veh/h}$$

Then, the capacity of the separate lane condition c_{sep} for each approach can be computed according to Equation 20-62:

$$c_{sep} = \min\left[c_R\left(1+\frac{v_{L+TH}}{v_R}\right), c_{L+TH}\left(1+\frac{v_R}{v_{L+TH}}\right)\right]$$

$$c_{sep,NB} = \min\left[c_{m,9}\left(1+\frac{v_{L+TH,NB}}{v_9}\right), c_{L+TH,NB}\left(1+\frac{v_9}{v_{L+TH,NB}}\right)\right]$$

$$c_{sep,NB} = \min\left[(845)\left(1+\frac{44+132}{55}\right),(385)\left(1+\frac{55}{44+132}\right)\right] = 505 \text{ veh/h}$$

$$c_{sep,SB} = \min\left[c_{m,12}\left(1+\frac{v_{L+TH,SB}}{v_{12}}\right), c_{L+TH,SB}\left(1+\frac{v_{12}}{v_{L+TH,SB}}\right)\right]$$

$$c_{sep,SB} = \min\left[(783)\left(1+\frac{11+110}{28}\right),(399)\left(1+\frac{28}{11+110}\right)\right] = 491 \text{ veh/h}$$

Finally, the capacities of the flared minor-street lanes are computed according to Equation 20-63:

$$c_R = \begin{cases} (c_{sep}-c_{SH})\frac{n_R}{n_{Max}} + c_{SH} & \text{if } n_R \le n_{Max} \\ c_{sep} & \text{if } n_R > n_{Max} \end{cases}$$

Because $n_R = 1$ and $n_{Max} = 2$, the first condition is evaluated:

$$c_{R,NB} = (505-442)\left(\frac{1}{2}\right) + 442 = 474 \text{ veh/h}$$

Similarly,

$$c_{R,SB} = (491-439)\left(\frac{1}{2}\right) + 439 = 465 \text{ veh/h}$$

Step 11: Compute Control Delay

The control delay computation for any movement includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The control delays for the major-street left-turn movements (Rank 2) d_1 and d_4 and the minor-street approaches d_{NB} and d_{SB} are computed with Equation 20-64:

$$d_1 = \frac{3{,}600}{1{,}100} + 900(0.25)\left[\frac{33}{1{,}100} - 1 + \sqrt{\left(\frac{33}{1{,}100} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}100}\right)\left(\frac{33}{1{,}100}\right)}{450(0.25)}}\right] + 5$$

$$d_1 = 8.4 \text{ s}$$

$$d_4 = \frac{3{,}600}{1{,}202} + 900(0.25)\left[\frac{66}{1{,}202} - 1 + \sqrt{\left(\frac{66}{1{,}202} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}202}\right)\left(\frac{66}{1{,}202}\right)}{450(0.25)}}\right] + 5$$

$$d_4 = 8.2 \text{ s}$$

$$d_{NB} = \frac{3{,}600}{474} + 900(0.25)\left[\frac{231}{474} - 1 + \sqrt{\left(\frac{231}{474} - 1\right)^2 + \frac{\left(\frac{3{,}600}{474}\right)\left(\frac{231}{474}\right)}{450(0.25)}}\right] + 5$$

$$d_{NB} = 19.6 \text{ s}$$

$$d_{SB} = \frac{3{,}600}{465} + 900(0.25)\left[\frac{149}{465} - 1 + \sqrt{\left(\frac{149}{465} - 1\right)^2 + \frac{\left(\frac{3{,}600}{465}\right)\left(\frac{149}{465}\right)}{450(0.25)}}\right] + 5$$

$$d_{SB} = 16.3 \text{ s}$$

According to Exhibit 20-2, LOS for the major-street left-turn movements and the minor-street approaches are as follows:

- Eastbound major-street left turn (Movement 1): LOS A,
- Westbound major-street left turn (Movement 4): LOS A,
- Northbound minor-street approach: LOS C, and
- Southbound minor-street approach: LOS C.

Step 11b: Compute Control Delay to Rank 1 Movements

This step is not applicable as the major-street through movements v_2 and v_5 and westbound major-street left-turn movements v_1 and v_4 have exclusive lanes at this intersection.

Step 12: Compute Approach and Intersection Control Delay

The control delay for the eastbound approach $d_{A,EB}$ is computed with Equation 20-66:

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

$$d_{A,EB} = \frac{0(50) + 0(250) + 8.2(33)}{50 + 250 + 33} = 0.8 \text{ s}$$

The control delay for the westbound approach $d_{A,WB}$ is computed according to the same equation as for the eastbound approach:

$$d_{A,WB} = \frac{0(100) + 0(300) + 8.4(66)}{100 + 300 + 66} = 1.2 \text{ s}$$

The intersection delay d_I is computed from Equation 20-67:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB} + d_{A,SB} v_{A,SB}}{v_{A,EB} + v_{A,WB} + v_{A,NB} + v_{A,SB}}$$

$$d_I = \frac{0.8(333) + 1.2(466) + 19.6(231) + 16.3(149)}{333 + 466 + 231 + 149} = 6.6 \text{ s}$$

LOS is not defined for the intersection as a whole or for the major-street approaches.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for the major-street eastbound left-turn movement $Q_{95,1}$ is computed from Equation 20-68:

$$Q_{95,1} \approx 900T \left[\frac{v_1}{c_{m,1}} - 1 + \sqrt{\left(\frac{v_1}{c_{m,1}} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{c_{m,1}} \right) \left(\frac{v_x}{c_{m,1}} \right)}{150T}} \right] \left(\frac{c_{m,1}}{3{,}600} \right)$$

$$Q_{95,1} \approx 900(0.25) \left[\frac{33}{1{,}100} - 1 + \sqrt{\left(\frac{33}{1{,}100} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{1{,}100} \right) \left(\frac{33}{1{,}100} \right)}{150(0.25)}} \right] \left(\frac{1{,}100}{3{,}600} \right)$$

$$Q_{95,1} \approx 0.1 \text{ veh}$$

The result of 0.1 vehicles for the 95th percentile queue indicates a queue of more than one vehicle will occur very infrequently for the eastbound major-street left-turn movement.

The 95th percentile queue length for the major-street westbound left-turn movement $Q_{95,4}$ is computed as follows:

$$Q_{95,4} \approx 900(0.25) \left[\frac{66}{1{,}202} - 1 + \sqrt{\left(\frac{66}{1{,}202} - 1 \right)^2 + \frac{\left(\frac{3{,}600}{1{,}202} \right) \left(\frac{66}{1{,}202} \right)}{150(0.25)}} \right] \left(\frac{1{,}202}{3{,}600} \right)$$

$$Q_{95,4} \approx 0.2 \text{ veh}$$

The result of 0.2 vehicles for the 95th percentile queue indicates a queue of more than one vehicle will occur very infrequently for the westbound major-street left-turn movement.

The 95th percentile queue length for the northbound approach is computed by using the same formula, but similar to the control delay computation, the shared-lane volume and shared-lane capacity must be used.

$$Q_{95,NB} \approx 900(0.25)\left[\frac{231}{474} - 1 + \sqrt{\left(\frac{231}{474} - 1\right)^2 + \frac{\left(\frac{3,600}{474}\right)\left(\frac{231}{474}\right)}{150(0.25)}}\right]\left(\frac{474}{3,600}\right)$$

$$Q_{95,NB} \approx 2.6 \text{ veh}$$

The result of 2.6 vehicles for the 95th percentile queue indicates a queue of more than two vehicles will occur occasionally for the northbound approach.

The 95th percentile queue length for the southbound approach is computed by using the same formula, but similar to the control delay computation, the shared-lane volume and shared-lane capacity must be used.

$$Q_{95,SB} \approx 900(0.25)\left[\frac{149}{465} - 1 + \sqrt{\left(\frac{149}{465} - 1\right)^2 + \frac{\left(\frac{3,600}{465}\right)\left(\frac{149}{465}\right)}{150(0.25)}}\right]\left(\frac{465}{3,600}\right)$$

$$Q_{95,SB} \approx 1.4 \text{ veh}$$

The result of 1.4 vehicles for the 95th percentile queue indicates a queue of more than one vehicle will occur occasionally for the southbound approach.

Discussion

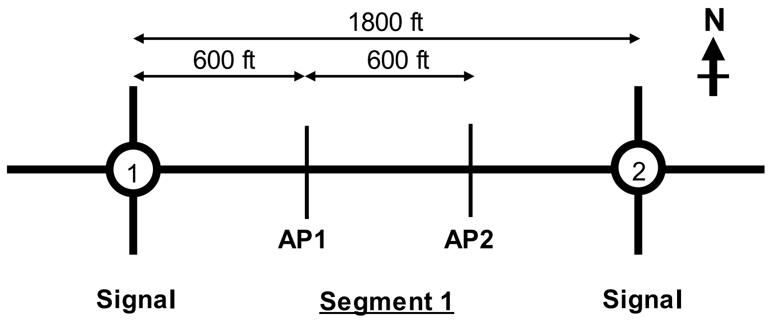
Overall, the results indicate the four-leg TWSC intersection with two-stage gap acceptance and flared minor-street approaches will operate satisfactorily with low delays for major-street movements and average delays for the minor-street approaches.

TWSC EXAMPLE PROBLEM 4: TWSC INTERSECTION WITHIN A SIGNALIZED URBAN STREET SEGMENT

The Facts

This problem analyzes the performance of the TWSC intersection at Access Point 1 (AP1) from Example Problem 1 in Chapter 30, Urban Street Segments: Supplemental, which looks at the motor vehicle performance of the urban street segment bounded by two signalized intersections, as shown in Exhibit 32-10. The street has a four-lane cross section with two lanes in each direction.

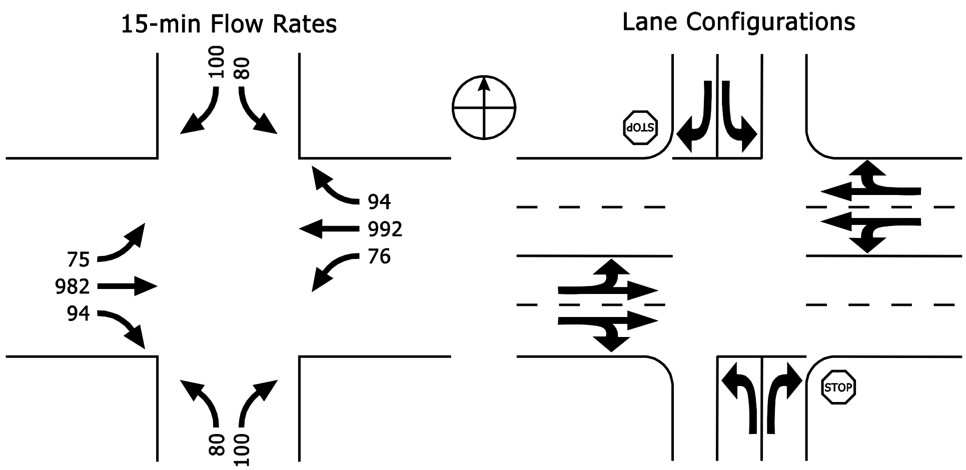
Exhibit 32-10
TWSC Example Problem 4: TWSC Intersection Within a Signalized Urban Street Segment



From Example Problem 1 in Chapter 30, the following data are relevant:

- Major street with two lanes in each direction,
- Minor street with separate left-turn and right-turn lanes in each direction (through movements considered negligible) and STOP control on minor-street approach,
- Level grade on all approaches,
- Percentage heavy vehicles on all approaches = 1%,
- Length of analysis period = 0.25 h, and
- Flow rates and lane configurations as shown in Exhibit 32-11.

Exhibit 32-11
TWSC Example Problem 4: 15-min Flow Rates and Lane Configurations



The proportion time blocked and delay to through vehicles from the methodology of Chapter 18, Urban Street Segments, are as shown in Exhibit 32-12.

Exhibit 32-12
TWSC Example Problem 4: Movement-Based Access Point Output (from Chapter 30, Example Problem 1)

Access Point Data Segment 1 Movement:	EB L 1	EB T 2	EB R 3	WB L 4	WB T 5	WB R 6	NB L 7	NB T 8	NB R 9	SB L 10	SB T 11	SB R 12
Access Point Intersection No. 1												
1: Volume, veh/h	74.80	981.71	93.50	75.56	991.70	94.45	80.00	0.00	100.00	80.00	0.00	100.00
1: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
1: Proportion time blocked	0.170			0.170			0.260	0.260	0.170	0.260	0.260	0.170
1: Delay to through vehicles, s/veh		0.163			0.164							
1: Prob. inside lane blocked by left		0.101			0.101							
1: Dist. from West/South signal, ft	600											
Access Point Intersection No. 2												
2: Volume, veh/h	75.56	991.70	94.45	74.80	981.71	93.50	80.00	0.00	100.00	80.00	0.00	100.00
2: Lanes	0	2	0	0	2	0	1	0	1	1	0	1
2: Proportion time blocked	0.170			0.170			0.260	0.260	0.170	0.260	0.260	0.170
2: Delay to through vehicles, s/veh		0.164			0.163							
2: Prob. inside lane blocked by left		0.101			0.101							
2: Dist. from West/South signal, ft	1200											

Comments

Default values are needed for the saturation flow rates of the major-street through and right-turn movements for the analysis of shared or short major-street left-turn lanes:

- Major-street through movement, s_{i1} = 1,800 veh/h; and
- Major-street right-turn movement, s_{i2} = 1,500 veh/h.

All other input parameters are known.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Flow rates for each turning movement have been provided from the methodology of Chapter 17, Urban Street Reliability and ATDM. They are assigned movement numbers as shown in Exhibit 32-13.

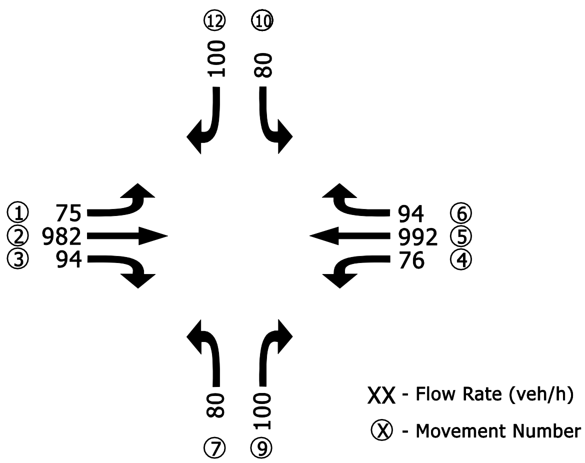


Exhibit 32-13
TWSC Example Problem 4: Movement Numbers and Calculation of Peak 15-min Flow Rates

Step 3: Compute Conflicting Flow Rates

Major-Street Left-Turn Movements (Rank 2, Movements 1 and 4)

The conflicting flows for the major-street left-turn movements are computed from Equation 20-2 and Equation 20-3 as follows:

$$v_{c,1} = v_5 + v_6 + v_{16} = 992 + 94 + 0 = 1{,}086 \text{ veh/h}$$

$$v_{c,4} = v_2 + v_3 + v_{15} = 982 + 94 + 0 = 1{,}076 \text{ veh/h}$$

Minor-Street Right-Turn Movements (Rank 2, Movements 9 and 12)

The conflicting flows for minor-street right-turn movements are computed from Equation 20-6 and Equation 20-7 as follows:

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15}$$

$$v_{c,9} = 0.5(982) + 0.5(94) + 0 + 0 + 0 = 538 \text{ veh/h}$$

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{1U} + v_{13} + v_{16}$$

$$v_{c,12} = 0.5(992) + 0.5(94) + 0 + 0 + 0 = 543 \text{ veh/h}$$

Major-Street U-Turn Movements (Rank 2, Movements 1U and 4U)

U-turns are assumed to be negligible.

Minor-Street Pedestrian Movements (Rank 2, Movements 13 and 14)

Minor-street pedestrian movements are assumed to be negligible.

Minor-Street Through Movements (Rank 3, Movements 8 and 11)

Because there are no minor-street through movements, this step can be skipped.

Minor-Street Left-Turn Movements (Rank 4, Movements 7 and 10)

Because the major street has four lanes without left-turn lanes or other possible median storage, the minor-street left-turn movement is assumed to be conducted in one stage. As a result, the conflicting flows for Stages I and II can be combined.

$$v_{c,7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15} + 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$

$$v_{c,7} = 2(75 + 0) + 982 + 0.5(94) + 0 + 2(76 + 0) + 0.5(992) + 0.5(0) + 0$$

$$v_{c,7} = 1{,}827 \text{ veh/h}$$

$$v_{c,10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16} + 2(v_1 + v_{1U}) + 0.5v_2 + 0.5v_8 + v_{14}$$

$$v_{c,10} = 2(76 + 0) + 992 + 0.5(94) + 0 + 2(75 + 0) + 0.5(982) + 0.5(0) + 0$$

$$v_{c,10} = 1{,}832 \text{ veh/h}$$

Step 4: Determine Critical Headways and Follow-Up Headways

Critical headways for each movement are computed from Equation 20-30:

$$t_{c,x} = t_{c,\text{base}} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,1} = t_{c,4} = 4.1 + (2.0)(0.01) + 0 - 0 = 4.12 \text{ s}$$

$$t_{c,9} = t_{c,12} = 6.9 + (2.0)(0.01) + 0.1(0) - 0 = 6.92 \text{ s}$$

$$t_{c,7} = t_{c,10} = 7.5 + (2.0)(0.01) + 0.2(0) - 0 = 7.52 \text{ s}$$

Follow-up headways for each movement are computed from Equation 20-31:

$$t_{f,x} = t_{f,base} + t_{f,HV}P_{HV}$$

$$t_{f,1} = t_{f,4} = 2.2 + (1.0)(0.01) = 2.21 \text{ s}$$

$$t_{f,9} = t_{f,12} = 3.3 + (1.0)(0.01) = 3.31 \text{ s}$$

$$t_{f,7} = t_{f,10} = 3.5 + (1.0)(0.01) = 3.51 \text{ s}$$

Step 5: Compute Potential Capacities

Because upstream signals are present, Step 5b is used. The proportion time blocked for each movement x is given as $p_{b,x}$ and has been computed by the Chapter 18 procedure.

The flow for the unblocked period (no platoons) is determined by first computing the conflicting flow for each movement during the unblocked period (Equation 20-33). The minimum platooned flow rate $v_{c,min}$ over two lanes is assumed to be equal to $1{,}000N = 1{,}000(2) = 2{,}000$. The flow rate assumed to occur during the blocked period is calculated as follows:

$$v_{c,u,x} = \begin{cases} \dfrac{v_{c,x} - 1.5 v_{c,min} p_{b,x}}{1 - p_{b,x}} & \text{if } v_{c,x} > 1.5 v_{c,min} p_{b,x} \\ 0 & \text{otherwise} \end{cases}$$

$$1.5 v_{c,min} p_{b,1} = 1.5(2{,}000)(0.170) = 510 \text{ veh/h}$$

The value for $v_{c,1}$ = 1,086 exceeds this value, which indicates some of the conflicting flow occurs in the unblocked period. Therefore, $v_{c,u,1}$ is calculated as follows:

$$v_{c,u,1} = \frac{v_{c,1} - 1.5 v_{c,min} p_{b,1}}{1 - p_{b,1}} = \frac{1{,}086 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 694 \text{ veh/h}$$

Similar calculations are made for the other movements:

$$v_{c,u,4} = \frac{1{,}076 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 682 \text{ veh/h}$$

$$v_{c,u,9} = \frac{538 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 34 \text{ veh/h}$$

$$v_{c,u,12} = \frac{543 - 1.5(2{,}000)(0.170)}{1 - 0.170} = 40 \text{ veh/h}$$

$$v_{c,u,7} = \frac{1{,}827 - 1.5(2{,}000)(0.260)}{1 - 0.260} = 1{,}415 \text{ veh/h}$$

$$v_{c,u,10} = \frac{1{,}832 - 1.5(2{,}000)(0.260)}{1 - 0.260} = 1{,}422 \text{ veh/h}$$

The potential capacity for each movement is then calculated with Equation 20-34 and Equation 20-35 (combined) as follows:

$$c_{p,1} = \left(1 - p_{b,1}\right)\left(v_{c,u,1}\right) \frac{e^{-v_{c,u,1} t_{c,1}/3{,}600}}{1 - e^{-v_{c,u,1} t_{f,1}/3{,}600}}$$

$$c_{p,1} = (1 - 0.170)(694) \frac{e^{-(694)(4.12)/3{,}600}}{1 - e^{-(694)(2.21)/3{,}600}} = 750 \text{ veh/h}$$

$$c_{p,4} = (1 - 0.170)(682) \frac{e^{-(682)(4.12)/3{,}600}}{1 - e^{-(682)(2.21)/3{,}600}} = 758 \text{ veh/h}$$

$$c_{p,9} = (1 - 0.170)(34) \frac{e^{-(34)(6.92)/3{,}600}}{1 - e^{-(34)(3.31)/3{,}600}} = 859 \text{ veh/h}$$

$$c_{p,12} = (1 - 0.170)(40) \frac{e^{-(40)(6.92)/3{,}600}}{1 - e^{-(40)(3.31)/3{,}600}} = 851 \text{ veh/h}$$

$$c_{p,7} = (1 - 0.260)(1{,}415) \frac{e^{-(1{,}415)(7.52)/3{,}600}}{1 - e^{-(1{,}415)(3.51)/3{,}600}} = 73 \text{ veh/h}$$

$$c_{p,10} = (1 - 0.260)(1{,}422) \frac{e^{-(1{,}422)(7.52)/3{,}600}}{1 - e^{-(1{,}422)(3.51)/3{,}600}} = 72 \text{ veh/h}$$

Steps 6–9: Compute Movement Capacities

Because no pedestrians are present, the procedures given in Chapter 20 are followed.

Step 6: Compute Rank 1 Movement Capacities

There is no computation for this step. The adjustment for the delay to through movements caused by left-turn movements in the shared left–through lane is accounted for by using adjustments provided later in this procedure.

Step 7: Compute Rank 2 Movement Capacities

Step 7a: Movement Capacity for Major-Street Left-Turn Movements

The movement capacity of each Rank 2 major-street left-turn movement is equal to its potential capacity as follows:

$$c_{m,1} = c_{p,1} = 750 \text{ veh/h}$$

$$c_{m,4} = c_{p,4} = 758 \text{ veh/h}$$

Step 7b: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity of each minor-street right-turn movement is equal to its potential capacity:

$$c_{m,9} = c_{p,9} = 859 \text{ veh/h}$$

$$c_{m,12} = c_{p,12} = 851 \text{ veh/h}$$

Step 7c: Movement Capacity for Major-Street U-Turn Movements

No U-turns are present, so this step is skipped.

Step 7d: Effect of Major-Street Shared Through and Left-Turn Lane

The probability that the major-street left-turning traffic will operate in a queue-free state, assuming the left-turn movement occupies its own lane, is calculated with Equation 20-42 as follows:

$$p_{0,1} = 1 - \frac{v_1}{c_{m,1}} = 1 - \frac{75}{750} = 0.900$$

$$p_{0,4} = 1 - \frac{v_4}{c_{m,4}} = 1 - \frac{76}{758} = 0.900$$

However, for this problem the major-street left-turn movement shares a lane with the through movement. First, the combined degree of saturation for the major-street through and right-turn movements is calculated as follows (using default values for s):

$$x_{2+3} = \frac{v_2}{s_2} + \frac{v_3}{s_3} = \frac{982}{1{,}800} + \frac{94}{1{,}500} = 0.608$$

$$x_{5+6} = \frac{v_5}{s_5} + \frac{v_6}{s_6} = \frac{992}{1{,}800} + \frac{94}{1{,}500} = 0.614$$

Next, the probability that there will be no queue in the major-street shared lane $p^*_{0,j}$ is calculated according to the special case ($n_L = 0$) given in Equation 20-45:

$$p_{0,1}^{*} = 1 - \frac{1 - p_{0,1}}{1 - x_{2+3}} = 1 - \frac{1 - 0.900}{1 - 0.608} = 0.745$$

$$p_{0,4}^{*} = 1 - \frac{1 - p_{0,4}}{1 - x_{5+6}} = 1 - \frac{1 - 0.900}{1 - 0.614} = 0.741$$

These values of $p_{0,1}^{*}$ and $p_{0,4}^{*}$ are used in lieu of $p_{0,1}$ and $p_{0,4}$ for the remaining calculations.

Step 8: Compute Rank 3 Movement Capacities

Step 8a: Rank 3 Capacity for One-Stage Movements

Because there are no minor-street through movements, it is not necessary to compute the movement capacities for those movements. However, capacity adjustment factors f_8 and f_{11} are needed for subsequent steps and can be computed as follows:

$$f_8 = f_{11} = p_{0,1}^{*} p_{0,4}^{*} = (0.745)(0.741) = 0.552$$

Step 8b: Rank 3 Capacity for Two-Stage Movements

No two-stage movements are present, so this step is skipped.

Step 9: Compute Rank 4 Movement Capacities

Step 9a: Rank 4 Capacity for One-Stage Movements

The probabilities that the minor-street right-turn movements will operate in the queue-free state $p_{0,9}$ and $p_{0,12}$ are computed as follows:

$$p_{0,9} = 1 - \frac{v_9}{c_{m,9}} = 1 - \frac{100}{859} = 0.884$$

$$p_{0,12} = 1 - \frac{v_{12}}{c_{m,12}} = 1 - \frac{100}{851} = 0.882$$

To compute p', the probability that both the major-street left-turn movements and the minor-street crossing movements will operate in a queue-free state simultaneously, the analyst must first compute $p_{0,k}$, which is done in the same manner as the computation of $p_{0,j}$, except k represents Rank 3 movements. The values for $p_{0,k}$ are computed as follows:

$$p_{0,8} = 1 - \frac{v_8}{c_{m,8}} = 1 - 0 = 1$$

$$p_{0,11} = 1 - \frac{v_{11}}{c_{m,11}} = 1 - 0 = 1$$

Next, the analyst must compute p'', which, under the single-stage gap-acceptance assumption, is simply the product of f_j and $p_{0,k}$. The value for $f_8 = f_{11} =$ 0.552 is as computed above. The value for $p_{0,11}$ is computed by using the total capacity for Movement 11 calculated in the previous step:

$$p_7'' = p_{0,11} \times f_{11} = (1)(0.552) = 0.552$$

$$p_{10}'' = p_{0,8} \times f_8 = (1)(0.552) = 0.552$$

By using the values for p'', the probability of a simultaneous queue-free state for each movement can be computed with Equation 20-52 as follows:

$$p'_7 = 0.65p''_7 - \frac{p''_7}{p''_7 + 3} + 0.6\sqrt{p''_7}$$

$$p'_7 = 0.65(0.552) - \frac{(0.552)}{0.552 + 3} + 0.6\sqrt{0.552} = 0.649$$

$$p'_{10} = 0.65(0.552) - \frac{(0.552)}{0.552 + 3} + 0.6\sqrt{0.552} = 0.649$$

Next, by using the probabilities computed above, capacity adjustment factors f_7 and f_{10} can be computed as follows:

$$f_7 = p'_7 \times p_{0,12} = (0.649)(0.882) = 0.572$$

$$f_{10} = p'_{10} \times p_{0,9} = (0.649)(0.884) = 0.574$$

Finally, the movement capacities $c_{m,7}$ and $c_{m,10}$ can be computed as follows:

$$c_{m,7} = c_{p,7} \times f_7 = (73)(0.572) = 42 \text{ veh/h}$$

$$c_{m,10} = c_{p,10} \times f_{10} = (72)(0.574) = 41 \text{ veh/h}$$

Step 9b: Rank 4 Capacity for Two-Stage Movements

No two-stage movements are present, so this step is skipped.

Step 10: Final Capacity Adjustments

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

No shared lanes are present on the side street, so this step is skipped.

Step 10b: Flared Minor-Street Lane Effects

No flared lanes are present, so this step is skipped.

Step 11: Compute Movement Control Delay

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The delay for each minor-street movement is calculated from Equation 20-64:

$$d_1 = \frac{3{,}600}{750} + 900(0.25)\left[\frac{75}{750} - 1 + \sqrt{\left(\frac{75}{750} - 1\right)^2 + \frac{\left(\frac{3{,}600}{750}\right)\left(\frac{75}{750}\right)}{450(0.25)}}\right] + 5$$

$$d_1 = 10.3 \text{ s}$$

$$d_4 = \frac{3{,}600}{758} + 900(0.25)\left[\frac{76}{758} - 1 + \sqrt{\left(\frac{76}{758} - 1\right)^2 + \frac{\left(\frac{3{,}600}{758}\right)\left(\frac{76}{758}\right)}{450(0.25)}}\right] + 5$$

$$d_4 = 10.3 \text{ s}$$

$$d_9 = \frac{3{,}600}{859} + 900(0.25)\left[\frac{100}{859} - 1 + \sqrt{\left(\frac{100}{859} - 1\right)^2 + \frac{\left(\frac{3{,}600}{859}\right)\left(\frac{100}{859}\right)}{450(0.25)}}\right] + 5$$

$$d_9 = 9.7 \text{ s}$$

$$d_{12} = \frac{3{,}600}{851} + 900(0.25)\left[\frac{100}{851} - 1 + \sqrt{\left(\frac{100}{851} - 1\right)^2 + \frac{\left(\frac{3{,}600}{851}\right)\left(\frac{100}{851}\right)}{450(0.25)}}\right] + 5$$

$$d_{12} = 9.8 \text{ s}$$

$$d_{7} = \frac{3{,}600}{42} + 900(0.25)\left[\frac{80}{42} - 1 + \sqrt{\left(\frac{80}{42} - 1\right)^2 + \frac{\left(\frac{3{,}600}{42}\right)\left(\frac{80}{42}\right)}{450(0.25)}}\right] + 5$$

$$d_{7} = 633 \text{ s}$$

$$d_{10} = \frac{3{,}600}{41} + 900(0.25)\left[\frac{80}{41} - 1 + \sqrt{\left(\frac{80}{41} - 1\right)^2 + \frac{\left(\frac{3{,}600}{41}\right)\left(\frac{80}{41}\right)}{450(0.25)}}\right] + 5$$

$$d_{10} = 657 \text{ s}$$

According to Exhibit 20-2, the LOS for the major-street left-turn movements and the minor-street approaches are as follows:

- Eastbound major-street left turn (Movement 1): LOS B,
- Westbound major-street left turn (Movement 4): LOS B,
- Northbound minor-street right turn (Movement 9): LOS A,
- Southbound minor-street right turn (Movement 12): LOS A,
- Northbound minor-street left turn (Movement 7): LOS F, and
- Southbound minor-street left turn (Movement 10): LOS F.

Step 11b: Compute Control Delay to Rank 1 Movements

The presence of a shared left–through lane on the major street creates delay for Rank 1 movements (major-street through movements). Assuming that major-street through vehicles distribute equally across both lanes, then $v_{i,1} = v_2/N = 982/2 = 491$. The number of major-street turning vehicles in the shared lane is equal to the major-street left-turn flow rate; therefore, $v_{i,2} = 75$.

The average delay to Rank 1 vehicles is computed with Equation 20-65 as follows:

$$d_{Rank1} = \begin{cases} \dfrac{\left(1 - p^*_{0,j}\right) d_{M,LT} \left(\frac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} & N > 1 \\ \left(1 - p^*_{0,j}\right) d_{M,LT} & N = 1 \end{cases}$$

$$d_2 = \frac{\left(1 - p^*_{0,1}\right) d_1 \left(\frac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} = \frac{(1 - 0.745)(10.3)\left(\frac{491}{2}\right)}{491 + 75} = 1.1 \text{ s}$$

Similarly, for the opposite direction, $v_{i,1} = v_5/N = 992/2 = 496$. The number of major-street turning vehicles in the shared lane is equal to the major-street left-turn flow rate; therefore, $v_{i,2} = 76$.

$$d_5 = \frac{\left(1 - p^*_{0,4}\right) d_4 \left(\frac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} = \frac{(1 - 0.741)(10.3)\left(\frac{496}{2}\right)}{496 + 76} = 1.2 \text{ s}$$

The procedures in Chapter 18 provide a better estimate of delay to major-street through vehicles: $d_2 = 0.2$ and $d_5 = 0.2$. These values account for the likelihood of major-street through vehicles shifting out of the shared left–through lane to avoid being delayed by major-street left-turning vehicles. These values are used in the calculations in Step 12.

Step 12: Compute Approach and Intersection Control Delay

The control delay for each approach is computed as follows:

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

$$d_{A,EB} = \frac{0(94) + 1.1(982) + 10.3(75)}{94 + 982 + 75} = 1.6 \text{ s}$$

$$d_{A,WB} = \frac{0(94) + 1.2(992) + 10.3(76)}{94 + 992 + 76} = 1.7 \text{ s}$$

$$d_{A,NB} = \frac{9.7(100) + 0 + 633(80)}{100 + 0 + 80} = 287 \text{ s}$$

$$d_{A,SB} = \frac{9.8(100) + 0 + 657(80)}{100 + 0 + 80} = 297 \text{ s}$$

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB} + d_{A,SB} v_{A,SB}}{v_{A,EB} + v_{A,WB} + v_{A,NB} + v_{A,SB}}$$

$$d_I = \frac{1.6(1{,}151) + 1.7(1{,}162) + 287(180) + 297(180)}{1{,}151 + 1{,}162 + 180 + 180} = 40.8 \text{ s}$$

LOS is not defined for the intersection as a whole or for the major-street approaches. This fact is particularly important for this problem, as the assignment of LOS to the intersection as a whole would mask the severe LOS F condition on the minor-street left-turn movement.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for each movement is computed by using Equation 20-68:

$$Q_{95,1} \approx 900T \left[\frac{v_1}{c_{m,1}} - 1 + \sqrt{\left(\frac{v_1}{c_{m,1}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,1}}\right)\left(\frac{v_1}{c_{m,1}}\right)}{150T}} \right] \left(\frac{c_{m,1}}{3{,}600}\right)$$

$$Q_{95,1} \approx 900(0.25) \left[\frac{75}{750} - 1 + \sqrt{\left(\frac{75}{750} - 1\right)^2 + \frac{\left(\frac{3{,}600}{750}\right)\left(\frac{75}{750}\right)}{150(0.25)}} \right] \left(\frac{750}{3{,}600}\right)$$

$$Q_{95,1} \approx 0.3 \text{ veh}$$

$$Q_{95,4} \approx 900(0.25)\left[\frac{76}{758} - 1 + \sqrt{\left(\frac{76}{758} - 1\right)^2 + \frac{\left(\frac{3,600}{758}\right)\left(\frac{76}{758}\right)}{150(0.25)}}\right]\left(\frac{758}{3,600}\right)$$

$$Q_{95,4} \approx 0.3 \text{ veh}$$

$$Q_{95,9} \approx 900(0.25)\left[\frac{100}{859} - 1 + \sqrt{\left(\frac{100}{859} - 1\right)^2 + \frac{\left(\frac{3,600}{859}\right)\left(\frac{100}{859}\right)}{150(0.25)}}\right]\left(\frac{859}{3,600}\right)$$

$$Q_{95,9} \approx 0.4 \text{ veh}$$

$$Q_{95,12} \approx 900(0.25)\left[\frac{100}{851} - 1 + \sqrt{\left(\frac{100}{851} - 1\right)^2 + \frac{\left(\frac{3,600}{851}\right)\left(\frac{100}{851}\right)}{150(0.25)}}\right]\left(\frac{851}{3,600}\right)$$

$$Q_{95,12} \approx 0.4 \text{ veh}$$

$$Q_{95,7} \approx 900(0.25)\left[\frac{80}{42} - 1 + \sqrt{\left(\frac{80}{42} - 1\right)^2 + \frac{\left(\frac{3,600}{42}\right)\left(\frac{80}{42}\right)}{150(0.25)}}\right]\left(\frac{42}{3,600}\right)$$

$$Q_{95,7} \approx 8.3 \text{ veh}$$

$$Q_{95,10} \approx 900(0.25)\left[\frac{80}{41} - 1 + \sqrt{\left(\frac{80}{41} - 1\right)^2 + \frac{\left(\frac{3,600}{41}\right)\left(\frac{80}{41}\right)}{150(0.25)}}\right]\left(\frac{41}{3,600}\right)$$

$$Q_{95,10} \approx 8.4 \text{ veh}$$

The results indicate that queues of more than one vehicle will rarely occur for the major-street left-turn and minor-street right-turn movements. Longer queues are expected for the minor-street left-turn movements, and these queues are likely to be unstable under the significantly oversaturated conditions.

Discussion

The results indicate that Access Point 1 will operate over capacity (LOS F) for the minor-street left-turn movements. All other movements are expected to operate at LOS B or better, with low average delays and short queue lengths.

TWSC EXAMPLE PROBLEM 5: SIX-LANE STREET WITH U-TURNS AND PEDESTRIANS

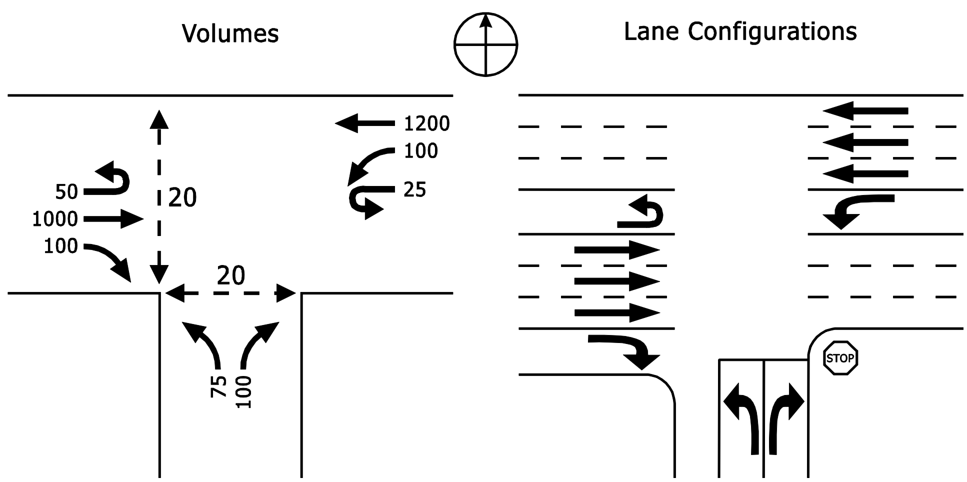
The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- T-intersection,
- Major street with three lanes in each direction,

- Minor street with separate left-turn and right-turn lanes and STOP control on the minor-street approach (minor-street left turns operate in two stages with room for storage of one vehicle),
- Level grade on all approaches,
- Percentage heavy vehicles on all approaches = 0%,
- Lane width = 12 ft,
- No other unique geometric considerations or upstream signal considerations,
- 20 p/h crossing both the west and south legs [each pedestrian is assumed to cross in his or her own group (i.e., independently)],
- Peak hour factor = 1.00,
- Length of analysis period = 0.25 h, and
- Hourly volumes and lane configurations as shown in Exhibit 32-14.

Exhibit 32-14
TWSC Example Problem 5: Volumes and Lane Configurations



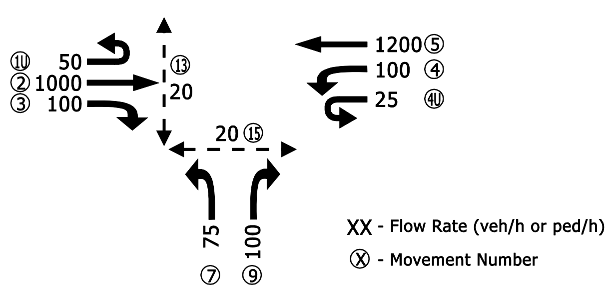
Comments

The assumed walking speed of pedestrians is 3.5 ft/s.

Steps 1 and 2: Convert Movement Demand Volumes to Flow Rates and Label Movement Priorities

Flow rates for each turning movement are the same as the peak hour volumes because the peak hour factor equals 1.0. These movements are assigned numbers as shown in Exhibit 32-15.

Exhibit 32-15
TWSC Example Problem 5: Movement Numbers and Calculation of Peak 15-min Flow Rates



Step 3: Compute Conflicting Flow Rates

Major-Street Left-Turn Movement (Rank 2, Movement 4)

The conflicting flow rate for the major-street left-turn movement is computed as follows:

$$v_{c,4} = v_2 + v_3 + v_{15} = 1{,}000 + 100 + 20 = 1{,}120 \text{ veh/h}$$

Minor-Street Right-Turn Movement (Rank 2, Movement 9)

The conflicting flow rate for the minor-street right-turn movement is computed as follows (dropping the v_3 term due to a separate major-street right-turn lane):

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{4U} + v_{14} + v_{15}$$

$$v_{c,9} = 0.5(1{,}000) + 0.5(0) + 0 + 0 + 20 = 520 \text{ veh/h}$$

Major-Street U-Turn Movements (Rank 2, Movements 1U and 4U)

The conflicting flow rates for the major-street U-turns are computed as follows (again dropping the v_3 term):

$$v_{c,1U} = 0.73v_5 + 0.73v_6 = 0.73(1{,}200) + 0 = 876 \text{ veh/h}$$

$$v_{c,4U} = 0.73v_2 + 0.73v_3 = 0.73(1{,}000) + 0 = 730 \text{ veh/h}$$

Minor-Street Left-Turn Movements (Rank 3, Movement 7)

The conflicting flow rate for Stage I of the minor-street left-turn movement is computed as follows (the v_3 term in these equations is assumed to be zero because of the right-turn lane on the major street):

$$v_{c,\mathrm{I},7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

$$v_{c,\mathrm{I},7} = 2(0 + 50) + 1{,}000 + 0 + 20 = 1{,}120 \text{ veh/h}$$

The conflicting flow rate for Stage II of the minor-street left-turn movement is computed as follows:

$$v_{c,\mathrm{II},7} = 2(v_4 + v_{4U}) + 0.4v_5 + 0.5v_{11} + v_{13}$$

$$v_{c,\mathrm{II},7} = 2(100 + 25) + 0.4(1{,}200) + 0 + 20 = 750 \text{ veh/h}$$

$$v_{c,7} = v_{c,\mathrm{I},7} + v_{c,\mathrm{II},7} = 1{,}120 + 750 = 1{,}870 \text{ veh/h}$$

Step 4: Determine Critical Headways and Follow-Up Headways

Critical headways for each minor movement are computed as follows:

$$t_{c,x} = t_{c,\text{base}} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,1U} = 5.6 + 0 + 0 - 0 = 5.6 \text{ s}$$

$$t_{c,4} = 5.3 + 0 + 0 - 0 = 5.3 \text{ s}$$

$$t_{c,4U} = 5.6 + 0 + 0 - 0 = 5.6 \text{ s}$$

$$t_{c,9} = 7.1 + 0 + 0 - 0 = 7.1 \text{ s}$$

$$t_{c,7} = 6.4 + 0 + 0 - 0.7 = 5.7 \text{ s}$$

$$t_{c,I,7} = 7.3 + 0 + 0 - 0.7 = 6.6 \text{ s}$$

$$t_{c,II,7} = 6.7 + 0 + 0 - 0.7 = 6.0 \text{ s}$$

Follow-up headways for each minor movement are computed as follows:

$$t_{f,x} = t_{f,\text{base}} + t_{f,HV} P_{HV}$$

$$t_{f,1U} = 2.3 + 0 = 2.3 \text{ s}$$

$$t_{f,4} = 3.1 + 0 = 3.1 \text{ s}$$

$$t_{f,4U} = 2.3 + 0 = 2.3 \text{ s}$$

$$t_{f,9} = 3.9 + 0 = 3.9 \text{ s}$$

$$t_{f,7} = 3.8 + 0 = 3.8 \text{ s}$$

Step 5: Compute Potential Capacities

Because no upstream signals are present, Step 5a is used. The potential capacity $c_{p,x}$ for each movement is computed as follows:

$$c_{p,x} = v_{c,x} \frac{e^{-v_{c,x} t_{c,x}/3{,}600}}{1 - e^{-v_{c,x} t_{f,x}/3{,}600}}$$

$$c_{p,1U} = v_{c,1U} \frac{e^{-v_{c,1U} t_{c,1U}/3{,}600}}{1 - e^{-v_{c,1U} t_{f,1U}/3{,}600}} = 876 \frac{e^{-(876)(5,6)/3{,}600}}{1 - e^{-(876)(2.3)/3{,}600}} = 523 \text{ veh/h}$$

$$c_{p,4} = 1{,}120 \frac{e^{-(1{,}120)(5,3)/3{,}600}}{1 - e^{-(1{,}120)(3.1)/3{,}600}} = 348 \text{ veh/h}$$

$$c_{p,4U} = 730 \frac{e^{-(730)(5,6)/3{,}600}}{1 - e^{-(730)(2.3)/3{,}600}} = 629 \text{ veh/h}$$

$$c_{p,9} = 520 \frac{e^{-(520)(7.1)/3{,}600}}{1 - e^{-(520)(3.9)/3{,}600}} = 433 \text{ veh/h}$$

$$c_{p,7} = 1{,}870 \frac{e^{-(1{,}870)(5,7)/3{,}600}}{1 - e^{-(1{,}870)(3.8)/3{,}600}} = 112 \text{ veh/h}$$

$$c_{p,I,7} = 1{,}120 \frac{e^{-(1{,}120)(6.6)/3{,}600}}{1 - e^{-(1{,}120)(3.8)/3{,}600}} = 207 \text{ veh/h}$$

$$c_{p,II,7} = 750 \frac{e^{-(750)(6.0)/3{,}600}}{1 - e^{-(750)(3.8)/3{,}600}} = 393 \text{ veh/h}$$

Steps 6–9: Compute Movement Capacities

Because of the presence of pedestrians, the computation steps provided earlier in this chapter should be used.

Step 6: Compute Rank 1 Movement Capacities

The methodology assumes Rank 1 vehicles are unimpeded by pedestrians.

Step 7: Compute Rank 2 Movement Capacities

Step 7a: Pedestrian Impedance

The factor accounting for pedestrian blockage is computed by Equation 20-69 as follows:

$$f_{pb} = \frac{v_x \times \frac{w}{S_p}}{3{,}600}$$

$$f_{pb,13} = \frac{v_{13} \times \frac{w}{S_p}}{3{,}600} = \frac{20 \times \frac{12}{3.5}}{3{,}600} = 0.019$$

$$f_{pb,15} = \frac{20 \times \frac{12}{3.5}}{3{,}600} = 0.019$$

The pedestrian impedance factor for each pedestrian movement x, $p_{p,x}$ is computed by Equation 20-70 as follows:

$$p_{p,13} = 1 - f_{pb,13} = 1 - 0.019 = 0.981$$
$$p_{p,15} = 1 - f_{pb,15} = 1 - 0.019 = 0.981$$

Step 7b: Movement Capacity for Major-Street Left-Turn Movements

On the basis of Exhibit 20-18, vehicular Movement 4 is impeded by pedestrian Movement 15. Therefore, the movement capacity for Rank 2 major-street left-turn movements is computed as follows:

$$c_{m,4} = c_{p,4} \times p_{p,15} = (348)(0.981) = 341 \text{ veh/h}$$

Step 7c: Movement Capacity for Minor-Street Right-Turn Movements

The northbound minor-street right-turn movement (Movement 9) is impeded by one conflicting pedestrian movement: Movement 15.

$$f_9 = p_{p,15} = 0.981$$

The movement capacity is then computed as follows:

$$c_{m,9} = c_{p,9} \times f_9 = (433)(0.981) = 425 \text{ veh/h}$$

Step 7d: Movement Capacity for Major-Street U-Turn Movements

The eastbound U-turn is unimpeded by queues from any other movement. Therefore, $f_{1U} = 1$, and the movement capacity is computed as follows:

$$c_{m,1U} = c_{p,1U} \times f_{1U} = (523)(0.981) = 523 \text{ veh/h}$$

For the westbound U-turn, the movement capacity is found by first computing a capacity adjustment factor that accounts for the impeding effects of minor-street right turns as follows:

$$f_{4U} = p_{0,9} = 1 - \frac{v_9}{c_{m,9}} = 1 - \frac{100}{425} = 0.765$$

The movement capacity is therefore computed as follows:

$$c_{m,4U} = c_{p,4U} \times f_{4U} = (629)(0.765) = 481 \text{ veh/h}$$

Because the westbound left-turn and U-turn movements are conducted from the same lane, their shared-lane capacity is computed as follows:

$$c_{m,4+4U} = \frac{v_4 + v_{4U}}{\frac{v_4}{c_{m,4}} + \frac{v_{4U}}{c_{m,4U}}} = \frac{100 + 25}{\frac{100}{341} + \frac{25}{481}} = 362 \text{ veh/h}$$

Step 7e: Effect of Major-Street Shared Through and Left-Turn Lane

This step is skipped.

Step 8: Compute Rank 3 Movement Capacities

There are no minor-street through movements, so the minor-street left-turn movement is treated as a Rank 3 movement.

Step 8a: Pedestrian Impedance

The northbound minor-street left turn (Movement 7) must yield to pedestrian Movements 13 and 15. Therefore, the impedance factor for pedestrians is as follows:

$$p_{p,7} = p_{p,15} \times p_{p,13} = (0.981)(0.981) = 0.962$$

Step 8b: Rank 3 Capacity for One-Stage Movements

The movement capacity $c_{m,k}$ for all Rank 3 movements is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements, assuming the movement operates in one stage. This value is computed as follows:

$$f_7 = p_{0,1U} \times p_{0,4+4U} \times p_{p,7} = \left(1 - \frac{v_{1U}}{c_{m,1U}}\right)\left(1 - \frac{v_{4+4U}}{c_{m,4+4U}}\right)(p_{p,7})$$

$$f_7 = \left(1 - \frac{50}{523}\right)\left(1 - \frac{100 + 25}{362}\right)(0.962) = 0.570$$

$$c_{m,7} = c_{p,7} \times f_7 = (112)(0.570) = 64 \text{ veh/h}$$

Step 8c: Rank 3 Capacity for Two-Stage Movements

Because the minor-street left-turn movement operates in two stages, the procedure for computing the total movement capacity for the subject movement considering the two-stage gap-acceptance process is followed.

First, the movement capacities for each stage of the left-turn movement are computed on the basis of the impeding movements for each stage. For Stage I, the left-turn movement is impeded by the major-street left and U-turns and by pedestrian Movement 15. Therefore,

$$f_{\mathrm{I},7} = p_{0,1U} \times p_{0,4+4U} \times p_{p,15} = \left(1 - \frac{v_{1U}}{c_{m,1U}}\right)\left(1 - \frac{v_{4+4U}}{c_{m,4+4U}}\right)(p_{p,15})$$

$$f_{\mathrm{I},7} = \left(1 - \frac{50}{523}\right)\left(1 - \frac{100 + 25}{362}\right)(0.981) = 0.581$$

$$c_{m,\mathrm{I},7} = c_{p,\mathrm{I},7} \times f_{\mathrm{I},7} = (207)(0.581) = 120 \text{ veh/h}$$

For Stage II, the left-turn movement is impeded only by pedestrian Movement 13. Therefore,

$$f_{\mathrm{II},7} = p_{p,13} = 0.981$$

$$c_{m,\mathrm{II},7} = c_{p,\mathrm{II},7} \times f_{\mathrm{II},7} = (393)(0.981) = 386 \text{ veh/h}$$

Next, an adjustment factor a and an intermediate variable y are computed for Movement 7 as follows:

$$a_7 = 1 - 0.32e^{-1.3\sqrt{n_m}} = 1 - 0.32e^{-1.3\sqrt{1}} = 0.913$$

$$y_7 = \frac{c_{m,I,7} - c_{m,7}}{c_{m,II,7} - v_{4+4U} - c_{m,7}} = \frac{120 - 64}{386 - 125 - 64} = 0.284$$

Therefore, the total capacity c_T is computed as follows:

$$c_{m,T,7} = \frac{a_7}{y_7^{n_m+1} - 1}\left[y_7\left(y_7^{n_m} - 1\right)\left(c_{m,II,7} - v_{4+4U}\right) + (y_7 - 1)c_{m,7}\right]$$

$$c_{m,T,7} = \frac{0.913}{0.284^{1+1} - 1}\left[(0.284)(0.284^1 - 1)(386 - 125) + (0.284 - 1)(64)\right]$$

$$c_{m,T,7} = 98 \text{ veh/h}$$

Step 9: Compute Rank 4 Movement Capacities

Because there are no Rank 4 movements, this step is skipped.

Step 10: Final Capacity Adjustments

There are no shared or flared lanes on the minor street, so this step is skipped.

Step 11: Compute Movement Control Delay

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

The control delay for each minor movement is computed as follows:

$$d_{1U} = \frac{3{,}600}{523} + 900(0.25)\left[\frac{50}{523} - 1 + \sqrt{\left(\frac{50}{523} - 1\right)^2 + \frac{\left(\frac{3{,}600}{523}\right)\left(\frac{50}{523}\right)}{450(0.25)}}\right] + 5$$

$$d_1 = 12.6 \text{ s}$$

This movement would be assigned LOS B.

$$d_{4+4U} = \frac{3{,}600}{362} + 900(0.25)\left[\frac{125}{362} - 1 + \sqrt{\left(\frac{125}{362} - 1\right)^2 + \frac{\left(\frac{3{,}600}{362}\right)\left(\frac{125}{362}\right)}{450(0.25)}}\right] + 5$$

$$d_{4+4U} = 20.1 \text{ s}$$

This movement would be assigned LOS C.

$$d_9 = \frac{3{,}600}{425} + 900(0.25)\left[\frac{100}{425} - 1 + \sqrt{\left(\frac{100}{425} - 1\right)^2 + \frac{\left(\frac{3{,}600}{425}\right)\left(\frac{100}{425}\right)}{450(0.25)}}\right] + 5$$

$$d_9 = 16.1 \text{ s}$$

This movement would be assigned LOS C.

$$d_7 = \frac{3{,}600}{98} + 900(0.25)\left[\frac{75}{98} - 1 + \sqrt{\left(\frac{75}{98} - 1\right)^2 + \frac{\left(\frac{3{,}600}{98}\right)\left(\frac{75}{98}\right)}{450(0.25)}}\right] + 5$$

$$d_1 = 113 \text{ s}$$

This movement would be assigned LOS F.

Step 11b: Compute Control Delay to Rank 1 Movements

No shared lanes are present on the major street, so this step is skipped.

Step 12: Compute Approach and Intersection Control Delay

The control delay for each approach is computed as follows:

$$d_{A,EB} = \frac{0(100) + 0(1{,}000) + 12.6(50)}{100 + 1{,}000 + 50} = 0.5 \text{ s}$$

$$d_{A,WB} = \frac{0(1{,}200) + 20.1(125)}{1{,}200 + 125} = 1.9 \text{ s}$$

$$d_{A,NB} = \frac{16.1(100) + 113(75)}{100 + 75} = 57.6 \text{ s}$$

The northbound approach is assigned LOS F. No LOS is assigned to the major-street approaches.

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB}v_{A,EB} + d_{A,WB}v_{A,WB} + d_{A,NB}v_{A,NB}}{v_{A,EB} + v_{A,WB} + v_{A,NB}}$$

$$d_I = \frac{0.5(1{,}150) + 1.9(1{,}325) + 57.6(175)}{1{,}150 + 1{,}325 + 175} = 5.0 \text{ s}$$

LOS is not defined for the intersection as a whole.

Step 13: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for each movement is computed from Equation 20-68:

$$Q_{95,1U} \approx 900(0.25)\left[\frac{50}{523} - 1 + \sqrt{\left(\frac{50}{523} - 1\right)^2 + \frac{\left(\frac{3{,}600}{523}\right)\left(\frac{50}{523}\right)}{150(0.25)}}\right]\left(\frac{523}{3{,}600}\right)$$

$$Q_{95,1} \approx 0.3 \text{ veh}$$

$$Q_{95,4+4U} \approx 900(0.25)\left[\frac{125}{362} - 1 + \sqrt{\left(\frac{125}{362} - 1\right)^2 + \frac{\left(\frac{3{,}600}{362}\right)\left(\frac{125}{362}\right)}{150(0.25)}}\right]\left(\frac{362}{3{,}600}\right)$$

$$Q_{95,4+4U} \approx 1.5 \text{ veh}$$

$$Q_{95,9} \approx 900(0.25)\left[\frac{100}{425} - 1 + \sqrt{\left(\frac{100}{425} - 1\right)^2 + \frac{\left(\frac{3{,}600}{425}\right)\left(\frac{100}{425}\right)}{150(0.25)}}\right]\left(\frac{425}{3{,}600}\right)$$

$$Q_{95,9} \approx 0.9 \text{ veh}$$

$$Q_{95,7} \approx 900(0.25)\left[\frac{75}{98} - 1 + \sqrt{\left(\frac{75}{98} - 1\right)^2 + \frac{\left(\frac{3{,}600}{98}\right)\left(\frac{75}{98}\right)}{150(0.25)}}\right]\left(\frac{98}{3{,}600}\right)$$

$$Q_{95,7} \approx 4.1 \text{ veh}$$

Discussion

Overall, the results indicate that although most minor movements are operating at low to moderate delays and at LOS C or better, the minor-street left turn experiences high delays and operates at LOS F.

4. AWSC SUPPLEMENTAL ANALYSIS FOR THREE-LANE APPROACHES

Exhibit 32-16 provides the 512 possible combinations of probability of degree-of-conflict cases when alternative lane occupancies are considered for three-lane approaches. A 1 indicates a vehicle is in the lane; a 0 indicates a vehicle is not in the lane.

Exhibit 32-16
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane)
(Cases 1–49)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
1	1	0	0	0	0	0	0	0	0	0	0
2	2	1	1	0	0	0	0	0	0	0	0
3			0	1	0	0	0	0	0	0	0
4			0	0	1	0	0	0	0	0	0
5		2	1	1	0	0	0	0	0	0	0
6			0	1	1	0	0	0	0	0	0
7			1	0	1	0	0	0	0	0	0
8		3	1	1	1	0	0	0	0	0	0
9	3	1	0	0	0	1	0	0	0	0	0
10			0	0	0	0	1	0	0	0	0
11			0	0	0	0	0	1	0	0	0
12			0	0	0	0	0	0	1	0	0
13			0	0	0	0	0	0	0	1	0
14			0	0	0	0	0	0	0	0	1
15		2	0	0	0	1	1	0	0	0	0
16			0	0	0	0	1	1	0	0	0
17			0	0	0	1	0	1	0	0	0
18			0	0	0	0	0	0	1	1	0
19			0	0	0	0	0	0	0	1	1
20			0	0	0	0	0	0	1	0	1
21		3	0	0	0	1	1	1	0	0	0
22			0	0	0	0	0	0	1	1	1
23	4	2	1	0	0	1	0	0	0	0	0
24			1	0	0	0	1	0	0	0	0
25			1	0	0	0	0	1	0	0	0
26			0	1	0	1	0	0	0	0	0
27			0	1	0	0	1	0	0	0	0
28			0	1	0	0	0	1	0	0	0
29			0	0	1	1	0	0	0	0	0
30			0	0	1	0	1	0	0	0	0
31			0	0	1	0	0	1	0	0	0
32			1	0	0	0	0	0	1	0	0
33			1	0	0	0	0	0	0	1	0
34			1	0	0	0	0	0	0	0	1
35			0	1	0	0	0	0	1	0	0
36			0	1	0	0	0	0	0	1	0
37			0	1	0	0	0	0	0	0	1
38			0	0	1	0	0	0	1	0	0
39			0	0	1	0	0	0	0	1	0
40			0	0	1	0	0	0	0	0	1
41			0	0	0	1	0	0	1	0	0
42			0	0	0	1	0	0	0	1	0
43			0	0	0	1	0	0	0	0	1
44			0	0	0	0	1	0	1	0	0
45			0	0	0	0	1	0	0	1	0
46			0	0	0	0	1	0	0	0	1
47			0	0	0	0	0	1	1	0	0
48			0	0	0	0	0	1	0	1	0
49			0	0	0	0	0	1	0	0	1

Exhibit 32-16 **(cont'd.)**
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane)
(Cases 50–112)

i	DOC Case	No. of Vehicles	Opposing Approach L1	Opposing Approach L2	Opposing Approach L3	Conflicting Left Approach L1	Conflicting Left Approach L2	Conflicting Left Approach L3	Conflicting Right Approach L1	Conflicting Right Approach L2	Conflicting Right Approach L3
50	4	3	1	1	0	1	0	0	0	0	0
51	(cont'd.)		1	1	0	0	1	0	0	0	0
52			1	1	0	0	0	1	0	0	0
53			0	1	1	1	0	0	0	0	0
54			0	1	1	0	1	0	0	0	0
55			0	1	1	0	0	1	0	0	0
56			1	0	1	1	0	0	0	0	0
57			1	0	1	0	1	0	0	0	0
58			1	0	1	0	0	1	0	0	0
59			1	1	0	0	0	0	1	0	0
60			1	1	0	0	0	0	0	1	0
61			1	1	0	0	0	0	0	0	1
62			0	1	1	0	0	0	1	0	0
63			0	1	1	0	0	0	0	1	0
64			0	1	1	0	0	0	0	0	1
65			1	0	1	0	0	0	1	0	0
66			1	0	1	0	0	0	0	1	0
67			1	0	1	0	0	0	0	0	1
68			1	0	0	1	1	0	0	0	0
69			1	0	0	0	1	1	0	0	0
70			1	0	0	1	0	1	0	0	0
71			0	1	0	1	1	0	0	0	0
72			0	1	0	0	1	1	0	0	0
73			0	1	0	1	0	1	0	0	0
74			0	0	1	1	1	0	0	0	0
75			0	0	1	0	1	1	0	0	0
76			0	0	1	1	0	1	0	0	0
77			0	0	0	1	1	0	1	0	0
78			0	0	0	1	1	0	0	1	0
79			0	0	0	1	1	0	0	0	1
80			0	0	0	0	1	1	1	0	0
81			0	0	0	0	1	1	0	1	0
82			0	0	0	0	1	1	0	0	1
83			0	0	0	1	0	1	1	0	0
84			0	0	0	1	0	1	0	1	0
85			0	0	0	1	0	1	0	0	1
86			1	0	0	0	0	0	1	1	0
87			1	0	0	0	0	0	0	1	1
88			1	0	0	0	0	0	1	0	1
89			0	1	0	0	0	0	1	1	0
90			0	1	0	0	0	0	0	1	1
91			0	1	0	0	0	0	1	0	1
92			0	0	1	0	0	0	1	1	0
93			0	0	1	0	0	0	0	1	1
94			0	0	1	0	0	0	1	0	1
95			0	0	0	1	0	0	1	1	0
96			0	0	0	1	0	0	0	1	1
97			0	0	0	1	0	0	1	0	1
98			0	0	0	0	1	0	1	1	0
99			0	0	0	0	1	0	0	1	1
100			0	0	0	0	1	0	1	0	1
101			0	0	0	0	0	1	1	1	0
102			0	0	0	0	0	1	0	1	1
103			0	0	0	0	0	1	1	0	1
104		4	1	1	0	1	1	0	0	0	0
105			1	1	0	0	1	1	0	0	0
106			1	1	0	1	0	1	0	0	0
107			0	1	1	1	1	0	0	0	0
108			0	1	1	0	1	1	0	0	0
109			0	1	1	1	0	1	0	0	0
110			1	0	1	1	1	0	0	0	0
111			1	0	1	0	1	1	0	0	0
112			1	0	1	1	0	1	0	0	0

Exhibit 32-16 **(cont'd.)**
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane)
(Cases 113–175)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
113	4	4	1	1	0	0	0	0	1	1	0
114	(cont'd.)	(cont'd.)	1	1	0	0	0	0	0	1	1
115			1	1	0	0	0	0	1	0	1
116			0	1	1	0	0	0	1	1	0
117			0	1	1	0	0	0	0	1	1
118			0	1	1	0	0	0	1	0	1
119			1	0	1	0	0	0	1	1	0
120			1	0	1	0	0	0	0	1	1
121			1	0	1	0	0	0	1	0	1
122			0	0	0	1	1	0	1	1	0
123			0	0	0	1	1	0	0	1	1
124			0	0	0	1	1	0	1	0	1
125			0	0	0	0	1	1	1	1	0
126			0	0	0	0	1	1	0	1	1
127			0	0	0	0	1	1	1	0	1
128			0	0	0	1	0	1	1	1	0
129			0	0	0	1	0	1	0	1	1
130			0	0	0	1	0	1	1	0	1
131			1	1	1	1	0	0	0	0	0
132			1	1	1	0	1	0	0	0	0
133			1	1	1	0	0	1	0	0	0
134			1	1	1	0	0	0	1	0	0
135			1	1	1	0	0	0	0	1	0
136			1	1	1	0	0	0	0	0	1
137			1	0	0	1	1	1	0	0	0
138			0	1	0	1	1	1	0	0	0
139			0	0	1	1	1	1	0	0	0
140			0	0	0	1	1	1	1	0	0
141			0	0	0	1	1	1	0	1	0
142			0	0	0	1	1	1	0	0	1
143			1	0	0	0	0	0	1	1	1
144			0	1	0	0	0	0	1	1	1
145			0	0	1	0	0	0	1	1	1
146			0	0	0	1	0	0	1	1	1
147			0	0	0	0	1	0	1	1	1
148			0	0	0	0	0	1	1	1	1
149		5	1	1	1	1	1	0	0	0	0
150			1	1	1	0	1	1	0	0	0
151			1	1	1	1	0	1	0	0	0
152			1	1	1	0	0	0	1	1	0
153			1	1	1	0	0	0	0	1	1
154			1	1	1	0	0	0	1	0	1
155			1	1	0	1	1	1	0	0	0
156			0	1	1	1	1	1	0	0	0
157			1	0	1	1	1	1	0	0	0
158			0	0	0	1	1	1	1	1	0
159			0	0	0	1	1	1	0	1	1
160			0	0	0	1	1	1	1	0	1
161			1	1	0	0	0	0	1	1	1
162			0	1	1	0	0	0	1	1	1
163			1	0	1	0	0	0	1	1	1
164			0	0	0	1	1	0	1	1	1
165			0	0	0	0	1	1	1	1	1
166			0	0	0	1	0	1	1	1	1
167		6	1	1	1	1	1	1	0	0	0
168			1	1	1	0	0	0	1	1	1
169			0	0	0	1	1	1	1	1	1
170	5	3	1	0	0	1	0	0	1	0	0
171			1	0	0	1	0	0	0	1	0
172			1	0	0	1	0	0	0	0	1
173			1	0	0	0	1	0	1	0	0
174			1	0	0	0	1	0	0	1	0
175			1	0	0	0	1	0	0	0	1

Exhibit 32-16 **(cont'd.)** Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 176–238)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
176	5	3	1	0	0	0	0	1	1	0	0
177	(cont'd.)	(cont'd.)	1	0	0	0	0	1	0	1	0
178			1	0	0	0	0	1	0	0	1
179			0	1	0	1	0	0	1	0	0
180			0	1	0	1	0	0	0	1	0
181			0	1	0	1	0	0	0	0	1
182			0	1	0	0	1	0	1	0	0
183			0	1	0	0	1	0	0	1	0
184			0	1	0	0	1	0	0	0	1
185			0	1	0	0	0	1	1	0	0
186			0	1	0	0	0	1	0	1	0
187			0	1	0	0	0	1	0	0	1
188			0	0	1	1	0	0	1	0	0
189			0	0	1	1	0	0	0	1	0
190			0	0	1	1	0	0	0	0	1
191			0	0	1	0	1	0	1	0	0
192			0	0	1	0	1	0	0	1	0
193			0	0	1	0	1	0	0	0	1
194			0	0	1	0	0	1	1	0	0
195			0	0	1	0	0	1	0	1	0
196			0	0	1	0	0	1	0	0	1
197		4	1	1	0	1	0	0	1	0	0
198			1	1	0	1	0	0	0	1	0
199			1	1	0	1	0	0	0	0	1
200			1	1	0	0	1	0	1	0	0
201			1	1	0	0	1	0	0	1	0
202			1	1	0	0	1	0	0	0	1
203			1	1	0	0	0	1	1	0	0
204			1	1	0	0	0	1	0	1	0
205			1	1	0	0	0	1	0	0	1
206			0	1	1	1	0	0	1	0	0
207			0	1	1	1	0	0	0	1	0
208			0	1	1	1	0	0	0	0	1
209			0	1	1	0	1	0	1	0	0
210			0	1	1	0	1	0	0	1	0
211			0	1	1	0	1	0	0	0	1
212			0	1	1	0	0	1	1	0	0
213			0	1	1	0	0	1	0	1	0
214			0	1	1	0	0	1	0	0	1
215			1	0	1	1	0	0	1	0	0
216			1	0	1	1	0	0	0	1	0
217			1	0	1	1	0	0	0	0	1
218			1	0	1	0	1	0	1	0	0
219			1	0	1	0	1	0	0	1	0
220			1	0	1	0	1	0	0	0	1
221			1	0	1	0	0	1	1	0	0
222			1	0	1	0	0	1	0	1	0
223			1	0	1	0	0	1	0	0	1
224			1	0	0	1	1	0	1	0	0
225			1	0	0	1	1	0	0	1	0
226			1	0	0	1	1	0	0	0	1
227			1	0	0	0	1	1	1	0	0
228			1	0	0	0	1	1	0	1	0
229			1	0	0	0	1	1	0	0	1
230			1	0	0	1	0	1	1	0	0
231			1	0	0	1	0	1	0	1	0
232			1	0	0	1	0	1	0	0	1
233			0	1	0	1	1	0	1	0	0
234			0	1	0	1	1	0	0	1	0
235			0	1	0	1	1	0	0	0	1
236			0	1	0	0	1	1	1	0	0
237			0	1	0	0	1	1	0	1	0
238			0	1	0	0	1	1	0	0	1

Exhibit 32-16 **(cont'd.)**
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane)
(Cases 239–301)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
239	5	4	0	1	0	1	0	1	1	0	0
240	(cont'd.)	(cont'd.)	0	1	0	1	0	1	0	1	0
241			0	1	0	1	0	1	0	0	1
242			0	0	1	1	1	0	1	0	0
243			0	0	1	1	1	0	0	1	0
244			0	0	1	1	1	0	0	0	1
245			0	0	1	0	1	1	1	0	0
246			0	0	1	0	1	1	0	1	0
247			0	0	1	0	1	1	0	0	1
248			0	0	1	1	0	1	1	0	0
249			0	0	1	1	0	1	0	1	0
250			0	0	1	1	0	1	0	0	1
251			1	0	0	1	0	0	1	1	0
252			1	0	0	1	0	0	0	1	1
253			1	0	0	1	0	0	1	0	1
254			1	0	0	0	1	0	1	1	0
255			1	0	0	0	1	0	0	1	1
256			1	0	0	0	1	0	1	0	1
257			1	0	0	0	0	1	1	1	0
258			1	0	0	0	0	1	0	1	1
259			1	0	0	0	0	1	1	0	1
260			0	1	0	1	0	0	1	1	0
261			0	1	0	1	0	0	0	1	1
262			0	1	0	1	0	0	1	0	1
263			0	1	0	0	1	0	1	1	0
264			0	1	0	0	1	0	0	1	1
265			0	1	0	0	1	0	1	0	1
266			0	1	0	0	0	1	1	1	0
267			0	1	0	0	0	1	0	1	1
268			0	1	0	0	0	1	1	0	1
269			0	0	1	1	0	0	1	1	0
270			0	0	1	1	0	0	0	1	1
271			0	0	1	1	0	0	1	0	1
272			0	0	1	0	1	0	1	1	0
273			0	0	1	0	1	0	0	1	1
274			0	0	1	0	1	0	1	0	1
275			0	0	1	0	0	1	1	1	0
276			0	0	1	0	0	1	0	1	1
277			0	0	1	0	0	1	1	0	1
278		5	1	1	0	1	1	0	1	0	0
279			1	1	0	1	1	0	0	1	0
280			1	1	0	1	1	0	0	0	1
281			1	1	0	0	1	1	1	0	0
282			1	1	0	0	1	1	0	1	0
283			1	1	0	0	1	1	0	0	1
284			1	1	0	1	0	1	1	0	0
285			1	1	0	1	0	1	0	1	0
286			1	1	0	1	0	1	0	0	1
287			0	1	1	1	1	0	1	0	0
288			0	1	1	1	1	0	0	1	0
289			0	1	1	1	1	0	0	0	1
290			0	1	1	0	1	1	1	0	0
291			0	1	1	0	1	1	0	1	0
292			0	1	1	0	1	1	0	0	1
293			0	1	1	1	0	1	1	0	0
294			0	1	1	1	0	1	0	1	0
295			0	1	1	1	0	1	0	0	1
296			1	0	1	1	1	0	1	0	0
297			1	0	1	1	1	0	0	1	0
298			1	0	1	1	1	0	0	0	1
299			1	0	1	0	1	1	1	0	0
300			1	0	1	0	1	1	0	1	0
301			1	0	1	0	1	1	0	0	1

Exhibit 32-15 (cont'd.) Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 302–364)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
302	5	5	1	0	1	1	0	1	1	0	0
303	(cont'd.)	(cont'd.)	1	0	1	1	0	1	0	1	0
304			1	0	1	1	0	1	0	0	1
305			1	1	0	1	0	0	1	1	0
306			1	1	0	0	1	0	1	1	0
307			1	1	0	0	0	1	1	1	0
308			1	1	0	1	0	0	0	1	1
309			1	1	0	0	1	0	0	1	1
310			1	1	0	0	0	1	0	1	1
311			1	1	0	1	0	0	1	0	1
312			1	1	0	0	1	0	1	0	1
313			1	1	0	0	0	1	1	0	1
314			0	1	1	1	0	0	1	1	0
315			0	1	1	0	1	0	1	1	0
316			0	1	1	0	0	1	1	1	0
317			0	1	1	1	0	0	0	1	1
318			0	1	1	0	1	0	0	1	1
319			0	1	1	0	0	1	0	1	1
320			0	1	1	1	0	0	1	0	1
321			0	1	1	0	1	0	1	0	1
322			0	1	1	0	0	1	1	0	1
323			1	0	1	1	0	0	1	1	0
324			1	0	1	0	1	0	1	1	0
325			1	0	1	0	0	1	1	1	0
326			1	0	1	1	0	0	0	1	1
327			1	0	1	0	1	0	0	1	1
328			1	0	1	0	0	1	0	1	1
329			1	0	1	1	0	0	1	0	1
330			1	0	1	0	1	0	1	0	1
331			1	0	1	0	0	1	1	0	1
332			1	0	0	1	1	0	1	1	0
333			1	0	0	1	1	0	0	1	1
334			1	0	0	1	1	0	1	0	1
335			1	0	0	0	1	1	1	1	0
336			1	0	0	0	1	1	0	1	1
337			1	0	0	0	1	1	1	0	1
338			1	0	0	1	0	1	1	1	0
339			1	0	0	1	0	1	0	1	1
340			1	0	0	1	0	1	1	0	1
341			0	1	0	1	1	0	1	1	0
342			0	1	0	1	1	0	0	1	1
343			0	1	0	1	1	0	1	0	1
344			0	1	0	0	1	1	1	1	0
345			0	1	0	0	1	1	0	1	1
346			0	1	0	0	1	1	1	0	1
347			0	1	0	1	0	1	1	1	0
348			0	1	0	1	0	1	0	1	1
349			0	1	0	1	0	1	1	0	1
350			0	0	1	1	1	0	1	1	0
351			0	0	1	1	1	0	0	1	1
352			0	0	1	1	1	0	1	0	1
353			0	0	1	0	1	1	1	1	0
354			0	0	1	0	1	1	0	1	1
355			0	0	1	0	1	1	1	0	1
356			0	0	1	1	0	1	1	1	0
357			0	0	1	1	0	1	0	1	1
358			0	0	1	1	0	1	1	0	1
359			1	1	1	1	0	0	1	0	0
360			1	1	1	1	0	0	0	1	0
361			1	1	1	1	0	0	0	0	1
362			1	1	1	0	1	0	1	0	0
363			1	1	1	0	1	0	0	1	0
364			1	1	1	0	1	0	0	0	1

Exhibit 32-15 **(cont'd.)**
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane)
(Cases 365–427)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
365	5	5	1	1	1	0	0	1	1	0	0
366	(cont'd.	(cont'd.)	1	1	1	0	0	1	0	1	0
367			1	1	1	0	0	1	0	0	1
368			1	0	0	1	1	1	1	0	0
369			1	0	0	1	1	1	0	1	0
370			1	0	0	1	1	1	0	0	1
371			0	1	0	1	1	1	1	0	0
372			0	1	0	1	1	1	0	1	0
373			0	1	0	1	1	1	0	0	1
374			0	0	1	1	1	1	1	0	0
375			0	0	1	1	1	1	0	1	0
376			0	0	1	1	1	1	0	0	1
377			1	0	0	1	0	0	1	1	1
378			1	0	0	0	1	0	1	1	1
379			1	0	0	0	0	1	1	1	1
380			0	1	0	1	0	0	1	1	1
381			0	1	0	0	1	0	1	1	1
382			0	1	0	0	0	1	1	1	1
383			0	0	1	1	0	0	1	1	1
384			0	0	1	0	1	0	1	1	1
385			0	0	1	0	0	1	1	1	1
386		6	1	1	0	1	1	0	1	1	0
387			1	1	0	1	1	0	0	1	1
388			1	1	0	1	1	0	1	0	1
389			1	1	0	0	1	1	1	1	0
390			1	1	0	0	1	1	0	1	1
391			1	1	0	0	1	1	1	0	1
392			1	1	0	1	0	1	1	1	0
393			1	1	0	1	0	1	0	1	1
394			1	1	0	1	0	1	1	0	1
395			0	1	1	1	1	0	1	1	0
396			0	1	1	1	1	0	0	1	1
397			0	1	1	1	1	0	1	0	1
398			0	1	1	0	1	1	1	1	0
399			0	1	1	0	1	1	0	1	1
400			0	1	1	0	1	1	1	0	1
401			0	1	1	1	0	1	1	1	0
402			0	1	1	1	0	1	0	1	1
403			0	1	1	1	0	1	1	0	1
404			1	0	1	1	1	0	1	1	0
405			1	0	1	1	1	0	0	1	1
406			1	0	1	1	1	0	1	0	1
407			1	0	1	0	1	1	1	1	0
408			1	0	1	0	1	1	0	1	1
409			1	0	1	0	1	1	1	0	1
410			1	0	1	1	0	1	1	1	0
411			1	0	1	1	0	1	0	1	1
412			1	0	1	1	0	1	1	0	1
413			1	1	1	1	1	0	1	0	0
414			1	1	1	1	1	0	0	1	0
415			1	1	1	1	1	0	0	0	1
416			1	1	1	0	1	1	1	0	0
417			1	1	1	0	1	1	0	1	0
418			1	1	1	0	1	1	0	0	1
419			1	1	1	1	0	1	1	0	0
420			1	1	1	1	0	1	0	1	0
421			1	1	1	1	0	1	0	0	1
422			1	1	1	1	0	0	1	1	0
423			1	1	1	1	0	0	0	1	1
424			1	1	1	1	0	0	1	0	1
425			1	1	1	0	1	0	1	1	0
426			1	1	1	0	1	0	0	1	1
427			1	1	1	0	1	0	1	0	1

	DOC	No. of	Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	Case	Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
428	5	6	1	1	1	0	0	1	1	1	0
429	(cont'd.)	(cont'd.)	1	1	1	0	0	1	0	1	1
430			1	1	1	0	0	1	1	0	1
431			1	1	0	1	1	1	1	0	0
432			1	1	0	1	1	1	0	1	0
433			1	1	0	1	1	1	0	0	1
434			0	1	1	1	1	1	1	0	0
435			0	1	1	1	1	1	0	1	0
436			0	1	1	1	1	1	0	0	1
437			1	0	1	1	1	1	1	0	0
438			1	0	1	1	1	1	0	1	0
439			1	0	1	1	1	1	0	0	1
440			1	0	0	1	1	1	1	1	0
441			1	0	0	1	1	1	0	1	1
442			1	0	0	1	1	1	1	0	1
443			0	1	0	1	1	1	1	1	0
444			0	1	0	1	1	1	0	1	1
445			0	1	0	1	1	1	1	0	1
446			0	0	1	1	1	1	1	1	0
447			0	0	1	1	1	1	0	1	1
448			0	0	1	1	1	1	1	0	1
449			1	1	0	1	0	0	1	1	1
450			1	1	0	0	1	0	1	1	1
451			1	1	0	0	0	1	1	1	1
452			0	1	1	1	0	0	1	1	1
453			0	1	1	0	1	0	1	1	1
454			0	1	1	0	0	1	1	1	1
455			1	0	1	1	0	0	1	1	1
456			1	0	1	0	1	0	1	1	1
457			1	0	1	0	0	1	1	1	1
458			1	0	0	1	1	0	1	1	1
459			1	0	0	0	1	1	1	1	1
460			1	0	0	1	0	1	1	1	1
461			0	1	0	1	1	0	1	1	1
462			0	1	0	0	1	1	1	1	1
463			0	1	0	1	0	1	1	1	1
464			0	0	1	1	1	0	1	1	1
465			0	0	1	0	1	1	1	1	1
466			0	0	1	1	0	1	1	1	1
467		7	1	1	1	1	1	0	1	1	0
468			1	1	1	1	1	0	0	1	1
469			1	1	1	1	1	0	1	0	1
470			1	1	1	0	1	1	1	1	0
471			1	1	1	0	1	1	0	1	1
472			1	1	1	0	1	1	1	0	1
473			1	1	1	1	0	1	1	1	0
474			1	1	1	1	0	1	0	1	1
475			1	1	1	1	0	1	1	0	1
476			1	1	0	1	1	1	1	1	0
477			1	1	0	1	1	1	0	1	1
478			1	1	0	1	1	1	1	0	1
479			0	1	1	1	1	1	1	1	0
480			0	1	1	1	1	1	0	1	1
481			0	1	1	1	1	1	1	0	1
482			1	0	1	1	1	1	1	1	0
483			1	0	1	1	1	1	0	1	1
484			1	0	1	1	1	1	1	0	1
485			1	1	0	1	1	0	1	1	1
486			1	1	0	0	1	1	1	1	1
487			1	1	0	1	0	1	1	1	1
488			0	1	1	1	1	0	1	1	1
489			0	1	1	0	1	1	1	1	1
490			0	1	1	1	0	1	1	1	1

Exhibit 32-15 **(cont'd.)**
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane) (Cases 428–490)

Exhibit 32-15 **(cont'd.)**
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Three-Lane Approaches, by Lane)
(Cases 491–512)

			Opposing Approach			Conflicting Left Approach			Conflicting Right Approach		
i	DOC Case	No. of Vehicles	L1	L2	L3	L1	L2	L3	L1	L2	L3
491	5	7	1	0	1	1	1	0	1	1	1
492	(cont'd.)	(cont'd.)	1	0	1	0	1	1	1	1	1
493			1	0	1	1	0	1	1	1	1
494			1	1	1	1	1	1	1	0	0
495			1	1	1	1	1	1	0	1	0
496			1	1	1	1	1	1	0	0	1
497			1	1	1	1	0	0	1	1	1
498			1	1	1	0	1	0	1	1	1
499			1	1	1	0	0	1	1	1	1
500			1	0	0	1	1	1	1	1	1
501			0	1	0	1	1	1	1	1	1
502			0	0	1	1	1	1	1	1	1
503		8	1	1	1	1	1	1	1	1	0
504			1	1	1	1	1	1	0	1	1
505			1	1	1	1	1	1	1	0	1
506			1	1	1	1	1	0	1	1	1
507			1	1	1	0	1	1	1	1	1
508			1	1	1	1	0	1	1	1	1
509			1	1	0	1	1	1	1	1	1
510			0	1	1	1	1	1	1	1	1
511			1	0	1	1	1	1	1	1	1
512		9	1	1	1	1	1	1	1	1	1

Note: DOC = degree-of-conflict; No. of vehicles = total number of vehicles on the opposing and conflicting approaches; L1, L2, and L3 = Lane 1, 2, and 3, respectively.

5. AWSC EXAMPLE PROBLEMS

This part of the chapter provides example problems for use of the AWSC methodology. Exhibit 32-17 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operations analysis level in terms of the calculations, except default values are used when available.

Exhibit 32-17
AWSC Example Problems

Problem Number	Description	Analysis Level
1	Single-lane, three-leg AWSC intersection	Operational
2	Multilane, four-leg AWSC intersection	Operational

AWSC EXAMPLE PROBLEM 1: SINGLE-LANE, THREE-LEG INTERSECTION

The Facts

The following describes this location's traffic and geometric characteristics:

- Three legs (T-intersection),
- One-lane entries on each leg,
- Percentage heavy vehicles on all approaches = 2%,
- Peak hour factor = 0.95, and
- Volumes and lane configurations are as shown in Exhibit 32-18.

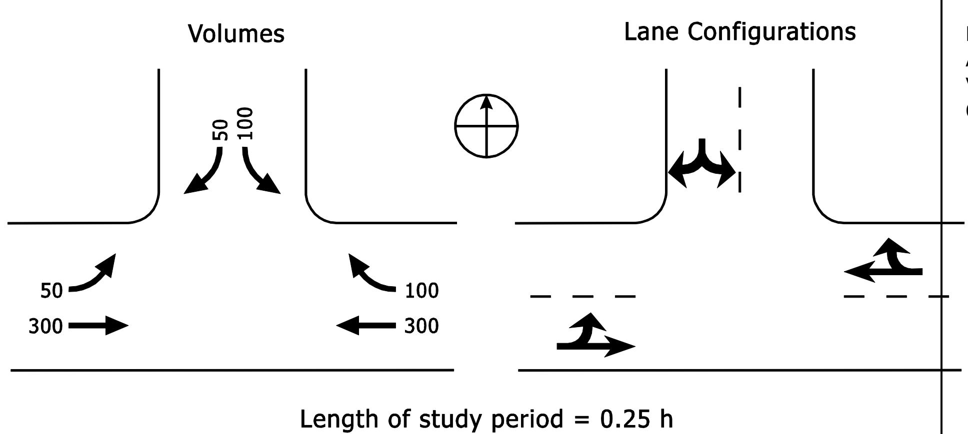


Exhibit 32-18
AWSC Example Problem 1: Volumes and Lane Configurations

Comments

All input parameters are known, so no default values are needed or used. The use of a spreadsheet or software is recommended because of the repetitive computations required. Slight differences in reported values may result from rounding differences between manual and software computations. Because showing all the individual computations is not practical, this example problem shows how one or more computations are made. All computational results can be found in the spreadsheet output located in the Volume 4 Technical Reference Library section for Chapter 32.

The use of a spreadsheet or software for AWSC intersection analysis is recommended because of the repetitive and iterative computations required.

Step 1: Convert Movement Demand Volumes to Flow Rates

Peak 15-min flow rates for each turning movement at the intersection are equal to the hourly volumes divided by the peak hour factor (Equation 21-12). For example, the peak 15-min flow rate for the eastbound through movement is as follows:

$$v_{EBTH} = \frac{V_{EBTH}}{PHF} = \frac{300}{0.95} = 316 \text{ veh/h}$$

Step 2: Determine Lane Flow Rates

This step does not apply because the intersection has one-lane approaches on all legs.

Step 3: Determine Geometry Group for Each Approach

Exhibit 21-11 shows each approach should be assigned to Geometry Group 1.

Step 4: Determine Saturation Headway Adjustments

Exhibit 21-12 shows the headway adjustments for left turns, right turns, and heavy vehicles are 0.2, –0.6, and 1.7, respectively. These values apply to all approaches because all are assigned to Geometry Group 1. The saturation headway adjustment for the eastbound approach is calculated from Equation 21-13 as follows:

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

$$h_{adj,EB} = 0.2\frac{53}{53+316} - 0.6(0) + 1.7(0.02) = 0.063$$

Similarly, the saturation headway adjustments for the westbound and northbound approaches are as follows:

$$h_{adj,WB} = 0.2(0) - 0.6\left(\frac{105}{105+316}\right) + 1.7(0.02) = -0.116$$

$$h_{adj,NB} = 0.2\frac{105}{105+53} - 0.6\left(\frac{53}{105+53}\right) + 1.7(0.02) = -0.034$$

Steps 5–11: Determine Departure Headways

These steps are iterative. The following narrative highlights some of the key calculations using the eastbound approach for Iteration 1.

Step 6: Calculate Initial Degree of Utilization

By using the lane flow rates from Step 2 and the assumed initial departure headway from Step 5, the initial degree of utilization x is computed as follows from Equation 21-14:

$$x_{EB} = \frac{vh_d}{3{,}600} = \frac{(368)(3.2)}{3{,}600} = 0.327$$

$$x_{WB} = \frac{(421)(3.2)}{3{,}600} = 0.374$$

$$x_{NB} = \frac{(158)(3.2)}{3{,}600} = 0.140$$

Step 7: Compute Probability States

The probability state of each combination i is determined with Equation 21-15.

$$P(i) = \prod_j P(a_j) = P(a_O) \times P(a_{CL}) \times P(a_{CR})$$

For an intersection with single-lane approaches, only eight cases from Exhibit 21-14 apply, as shown in Exhibit 32-19:

Exhibit 32-19
AWSC Example Problem 1: Applicable Degree-of-Conflict Cases

i	DOC Case	No. of Vehicles	Opposing Approach	Conflicting Left Approach	Conflicting Right Approach
1	1	0	0	0	0
2	2	1	1	0	0
5	3	1	0	1	0
7	3	1	0	0	1
13	4	2	0	1	1
16	4	2	1	1	0
21	4	2	1	0	1
45	5	3	1	1	1

For example, the probability state for the eastbound leg under the condition of no opposing vehicles on the other approaches (degree-of-conflict Case 1, $i = 1$) is as follows:

$P(a_O) = 1 - x_O = 1 - 0.374 = 0.626$ (no opposing vehicle present)

$P(a_{CL}) = 1 - x_{CL} = 1 - 0.140 = 0.860$ (no conflicting vehicle from left)

$P(a_{CR}) = 1$ (no approach conflicting from right)

Therefore,

$$P(1) = P(a_O) \times P(a_{CL}) \times P(a_{CR}) = (0.626)(0.860)(1) = 0.538$$

Similarly,

$$P(2) = (0.374)(0.860)(1) = 0.322$$
$$P(5) = (0.626)(0.140)(1) = 0.088$$
$$P(7) = (0.626)(0.860)(0) = 0$$
$$P(13) = (0.626)(0.140)(0) = 0$$
$$P(16) = (0.374)(0.140)(1) = 0.052$$
$$P(21) = (0.374)(0.860)(0) = 0$$
$$P(45) = (0.374)(0.140)(0) = 0$$

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed as follows, using Equation 21-16 through Equation 21-20:

$$P(C_1) = P(1) = 0.538$$
$$P(C_2) = P(2) = 0.322$$
$$P(C_3) = P(5) + P(7) = 0.088 + 0 = 0.088$$
$$P(C_4) = P(13) + P(16) + P(21) = 0 + 0.052 + 0 = 0.052$$
$$P(C_5) = P(45) = 0$$

The probability adjustment factors for the nonzero cases are calculated from Equation 21-21 through Equation 21-25:

$$AdjP(1) = 0.01[0.322 + 2(0.088) + 3(0.052) + 0]/1 = 0.0065$$

$$AdjP(2) = 0.01[0.088 + 2(0.052) + 0 - 0.322]/3 = -0.0004$$

$$AdjP(5) = 0.01[0.052 + 2(0) - 3(0.088)]/6 = -0.0004$$

$$AdjP(16) = 0.01[0 - 6(0.052)]/27 = -0.0001$$

Therefore, the adjusted probability for Combination 1, for example, is as follows from Equation 21-16:

$$P'(1) = P(1) + AdjP(1) = 0.538 + 0.0065 = 0.5445$$

Step 9: Compute Saturation Headways

The base saturation headways for each combination can be determined with Exhibit 21-15. They are adjusted by using the adjustment factors calculated in Step 4 and added to the base saturation headways to determine saturation headways as shown in Exhibit 32-20 (eastbound illustrated):

Exhibit 32-20
AWSC Example Problem 1: Eastbound Saturation Headways

i	h_{base}	h_{adj}	h_{si}
1	3.9	0.063	3.963
2	4.7	0.063	4.763
5	5.8	0.063	5.863
7	7.0	0.063	7.063

Step 10: Compute Departure Headways

The departure headway of the lane is the sum of the products of the adjusted probabilities and the saturation headways as follows (eastbound illustrated):

$$h_d = \sum_{i=1}^{64} P'(i) h_{si}$$

$$h_{d,EB} = (0.5445)(3.963) + (0.3213)(4.763) + (0.0875)(5.863) + (0.0524)(7.063)$$

$$h_{d,EB} = 4.57 \text{ s}$$

Step 11: Check for Convergence

The calculated values of h_d are checked against the initial values assumed for h_d. After one iteration, each calculated headway differs from the initial value by more than 0.1 s. Therefore, the new calculated headway values are used as initial values in a second iteration. For this problem, four iterations are required for convergence, as shown in Exhibit 32-21.

Exhibit 32-21
AWSC Example Problem 1: Convergence Check

	EB L1	EB L2	WB L1	WB L2	NB L1	NB L2	SB L1	SB L2
Total Lane Flow Rate	368		421				158	
hd, initial value, iteration 1	3.2		3.2				3.2	
x, initial, iteration 1	0.327		0.374				0.140	
hd, computed value, iteration 1	4.57		4.35				5.14	
Convergence?	N		N				N	
hd, initial value, iteration 2	4.57		4.35				5.14	
x, initial, iteration 2	0.468		0.509				0.225	
hd, computed value, iteration 2	4.88		4.66				5.59	
Convergence?	N		N				N	
hd, initial value, iteration 3	4.88		4.66				5.59	
x, initial, iteration 3	0.499		0.545				0.245	
hd, computed value, iteration 3	4.95		4.73				5.70	
Convergence?	Y		Y				N	
hd, initial value, iteration 4	4.88		4.66				5.70	
x, initial, iteration 4	0.499		0.545				0.250	
hd, computed value, iteration 4	4.97		4.74				5.70	
Convergence?	Y		Y				Y	

Step 12: Compute Capacities

The capacity of each lane in a subject approach is computed by increasing the given flow rate on the subject lane (assuming the flows on the opposing and conflicting approaches are constant) until the degree of utilization for the subject lane reaches 1. This level of calculation requires running an iterative procedure many times, which is practical for a spreadsheet or software implementation.

Here, the eastbound lane capacity is approximately 720 veh/h, which is lower than the value that could be estimated by dividing the lane volume by the degree of utilization (368/0.492 = 748 veh/h). The difference is due to the interaction effects among the approaches: increases in eastbound traffic volume increase the departure headways of the lanes on the other approaches, which in turn increases the departure headways of the lane(s) on the subject approach.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time by using Equation 21-29. For the eastbound lane (using a value for m of 2.0 for Geometry Group 1), the calculation is as follows:

$$t_{s,EB} = h_{d,EB} - m = 4.97 - 2.0 = 2.97 \text{ s}$$

Step 14: Compute Control Delay and Determine LOS for Each Lane

The control delay for each lane is computed with Equation 21-30 as follows (eastbound illustrated):

$$d_{EB} = t_{s,EB} + 900T\left[(x_{EB} - 1) + \sqrt{(x_{EB} - 1)^2 + \frac{h_{d,EB}x_{EB}}{450T}}\right] + 5$$

$$d_{EB} = 2.97 + 900(0.25)\left[(0.508 - 1) + \sqrt{(0.508 - 1)^2 + \frac{4.97(0.508)}{450(0.25)}}\right] + 5$$

$$d_{EB} = 13.0 \text{ s}$$

By using Exhibit 21-8, the eastbound lane (and thus approach) is assigned LOS B. A similar calculation for the westbound and southbound lanes (and thus approaches) yields 13.5 and 10.6 s, respectively.

Step 15: Compute Control Delay and Determine LOS for the Intersection

The control delays for the approaches can be combined into an intersection control delay by using a weighted average as follows:

$$d_{\text{intersection}} = \frac{\sum d_a v_a}{\sum v_a}$$

$$d_{\text{intersection}} = \frac{(13.0)(368) + (13.5)(421) + (10.6)(158)}{368 + 421 + 158} = 12.8 \text{ s}$$

This value of delay is assigned LOS B.

Step 16: Compute Queue Lengths

The 95th percentile queue for each lane is computed with Equation 21-33 as follows (eastbound approach illustrated):

$$Q_{95,EB} \approx \frac{900T}{h_{d,EB}} \left[(x_{EB} - 1) + \sqrt{(x_{EB} - 1)^2 + \frac{h_{d,EB} x_{EB}}{150T}} \right]$$

$$Q_{95,EB} \approx \frac{900(0.25)}{4.97} \left[(0.508 - 1) + \sqrt{(0.508 - 1)^2 + \frac{4.97(0.508)}{150(0.25)}} \right] = 2.9 \text{ veh}$$

This queue length would be reported as three vehicles.

Discussion

The results indicate the intersection operates well with brief delays.

AWSC EXAMPLE PROBLEM 2: MULTILANE, FOUR-LEG INTERSECTION

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Four legs;
- Two-lane approaches on the east and west legs;
- Three-lane approaches on the north and south legs;
- Percentage heavy vehicles on all approaches = 2%;
- Demand volumes are provided in 15-min intervals (therefore, a peak hour factor is not required), and the analysis period length is 0.25 h; and
- Volumes and lane configurations are as shown in Exhibit 32-22.

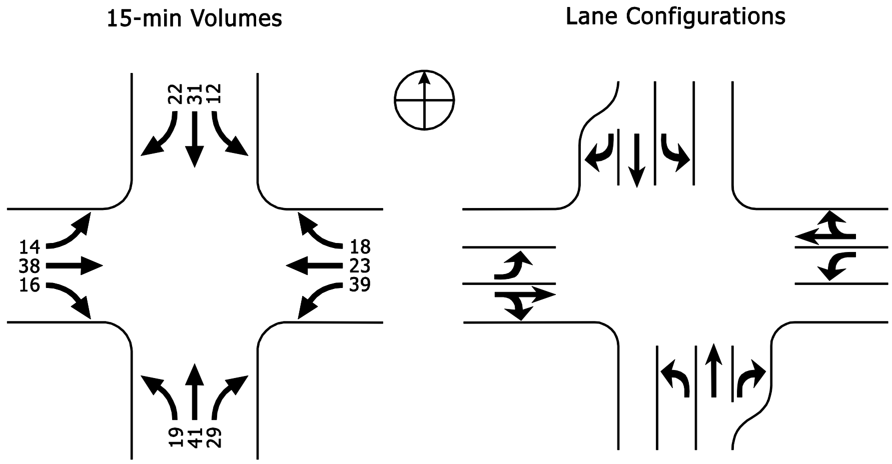


Exhibit 32-22
AWSC Example Problem 2: 15-min Volumes and Lane Configurations

Comments

All input parameters are known, so no default values are needed or used. The use of a spreadsheet or software is required because of the several thousand repetitive computations needed. Slight differences in reported values may result from rounding differences between manual and software computations. Because showing all the individual computations is not practical, this example problem shows how one or more computations are made. All computational results can be found in the spreadsheet output located in the Volume 4 Technical Reference Library section for Chapter 32.

Step 1: Convert Movement Demand Volumes to Flow Rates

To convert the peak 15-min demand volumes to hourly flow rates, the individual movement volumes are simply multiplied by four, as shown in Exhibit 32-23:

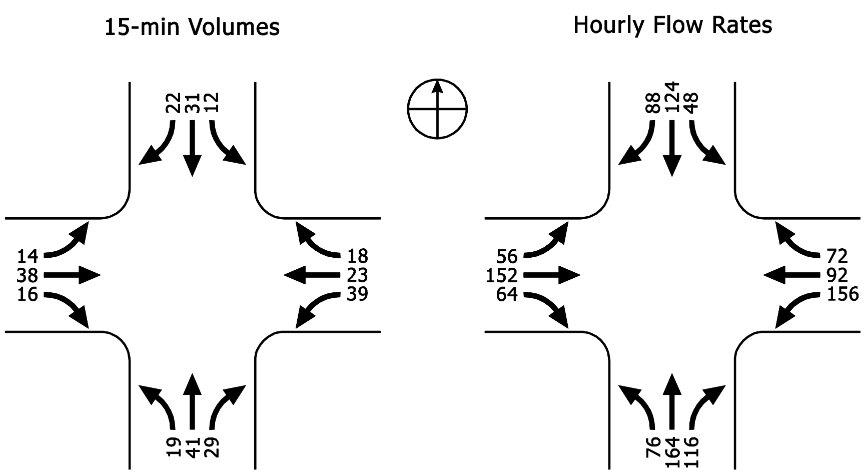


Exhibit 32-23
AWSC Example Problem 2: 15-min Volumes Converted to Hourly Flow Rates

Step 2: Determine Lane Flow Rates

This step simply involves assigning the turning movement volume to each of the approach lanes. The left-turn volume is assigned to the separate left-turn lane on each approach. For the east and west approaches, the through and right-turn volumes are assigned to the shared through and right lanes. For the north and south approaches, the through volumes are assigned to the through lanes and the right-turn volumes are assigned to the right-turn lanes.

Step 3: Determine Geometry Group for Each Approach

Exhibit 21-11 shows each approach should be assigned to Geometry Group 6.

Step 4: Determine Saturation Headway Adjustments

Exhibit 21-12 shows the headway adjustments for left turns, right turns, and heavy vehicles are 0.5, –0.7, and 1.7, respectively. These values apply to all approaches as all are assigned Geometry Group 6. The saturation headway adjustment for the eastbound approach is as follows for Lane 1 (the left-turn lane):

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

$$h_{adj,EB,1} = 0.5(1.0) - 0.7(0) + 1.7(0.02) = 0.534$$

Similarly, the saturation headway adjustment for Lane 2 of the eastbound approach is as follows:

$$h_{adj,EB,2} = 0.5(0) - 0.7\left(\frac{64}{64+152}\right) + 1.7(0.02) = -0.173$$

The saturation headway adjustment for all the remaining lanes by approach is similarly calculated. The full computational results can be seen in the "HdwyAdj" spreadsheet tab.

Steps 5–11: Determine Departure Headways

These steps are iterative and, for this example, involve several thousand calculations. The following narrative highlights some of the key calculations using the eastbound approach for Iteration 1, but it does not attempt to reproduce all calculations for all iterations. The full computational results for each of the iterative computations can be seen in the "DepHdwyIterX" spreadsheet tab, where "X" is the iteration.

Step 6: Calculate Initial Degree of Utilization

The remainder of this example illustrates the calculations needed to evaluate Lane 1 on the eastbound approach (eastbound left turn). Step 6 requires calculating the initial degree of utilization for all the opposing and conflicting lanes. They are computed as follows:

$$x_{WB,1} = \frac{vh_d}{3{,}600} = \frac{(156)(3.2)}{3{,}600} = 0.1387$$

$$x_{WB,2} = \frac{vh_d}{3{,}600} = \frac{(164)(3.2)}{3{,}600} = 0.1458$$

$$x_{NB,1} = \frac{vh_d}{3{,}600} = \frac{(76)(3.2)}{3{,}600} = 0.0676$$

$$x_{NB,2} = \frac{vh_d}{3{,}600} = \frac{(164)(3.2)}{3{,}600} = 0.1458$$

$$x_{NB,3} = \frac{vh_d}{3{,}600} = \frac{(116)(3.2)}{3{,}600} = 0.1031$$

$$x_{SB,1} = \frac{vh_d}{3{,}600} = \frac{(48)(3.2)}{3{,}600} = 0.0427$$

$$x_{SB,2} = \frac{vh_d}{3{,}600} = \frac{(124)(3.2)}{3{,}600} = 0.1102$$

$$x_{SB,3} = \frac{vh_d}{3{,}600} = \frac{(88)(3.2)}{3{,}600} = 0.0782$$

Step 7: Compute Probability States

Because three-lane approaches are involved, the modified methodology presented in Section 4 of Chapter 21 is used.

The probability state of each combination i is determined with Equation 21-34:

$$P(i) = \prod_j P(a_j) = P(a_O) \times P(a_{CL}) \times P(a_{CR})$$

For example, the probability state for the eastbound leg under the condition of no opposing vehicles on the other approaches (Degree-of-Conflict Case 1, $i = 1$) is as follows (using Exhibit 21-16):

$$P(a_{O1}) = 1 - x_{O1} = 1 - 0.1387 = 0.8613 \quad \text{(opposing westbound Lane 1)}$$

$$P(a_{O2}) = 1 - x_{O2} = 1 - 0.1458 = 0.8542 \quad \text{(opposing westbound Lane 2)}$$

$$P(a_{CL1}) = 1 - x_{CL1} = 1 - 0.0427 = 0.9573 \quad \text{(conflicting from left Lane 1)}$$

$$P(a_{CL2}) = 1 - x_{CL2} = 1 - 0.1102 = 0.8898 \quad \text{(conflicting from left Lane 2)}$$

$$P(a_{CL3}) = 1 - x_{CL3} = 1 - 0.0782 = 0.9218 \quad \text{(conflicting from left Lane 3)}$$

$$P(a_{CR1}) = 1 - x_{CR1} = 1 - 0.0676 = 0.9324 \quad \text{(conflicting from right Lane 1)}$$

$$P(a_{CR2}) = 1 - x_{CR2} = 1 - 0.1458 = 0.8542 \quad \text{(conflicting from right Lane 2)}$$

$$P(a_{CR3}) = 1 - x_{CR3} = 1 - 0.1031 = 0.8969 \quad \text{(conflicting from right Lane 3)}$$

Therefore,

$$P(1) = P(a_{O1}) \times P(a_{O2}) \times P(a_{CL1}) \times P(a_{CL2}) \times P(a_{CL3}) \times P(a_{CR1}) \times P(a_{CR2}) \times P(a_{CR3})$$

$$P(1) = (0.8613)(0.8542)(0.9573)(0.8898)(0.9218)(0.9324)(0.8542)(0.8969)$$

$$P(1) = 0.4127$$

To complete the calculations for Step 7, the computations are completed for the remaining 511 possible combinations. The full computational results for the eastbound leg (Lane 1) can be seen in the "DepHdwyIter1" spreadsheet tab, Rows 3118–3629 (Columns C–K).

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 21-35 through Equation 21-39 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined. For the example of eastbound Lane 1, these computations are made by summing Rows 3118–3629 in the spreadsheet for each of the five cases (Columns R–V). The resulting computations are shown in Row 3630 (Columns R–V), where

$$P(C_1) = P(1) = 0.4127$$

$$P(C_2) = \sum_{i=2}^{8} P(i) = 0.1482$$

$$P(C_3) = \sum_{i=9}^{22} P(i) = 0.2779$$

$$P(C_4) = \sum_{i=23}^{169} P(i) = 0.1450$$

$$P(C_5) = \sum_{i=170}^{512} P(i) = 0.0162$$

The probability adjustment factors are then computed with Equation 21-40 through Equation 21-44, where α equals 0.01 (or 0.00 if correlation among saturation headways is not taken into account).

For example, by using Equation 21-35, $AdjP(1)$ is calculated as follows:

$$AdjP(1) = 0.01[0.1482 + 2(0.2779) + 3(0.1450) + 4(0.0162)]/1 = 0.01204$$

The results of the remaining computations for eastbound Lane 1 are located in Row 3632 of the spreadsheet (Columns S–V).

Step 9: Compute Saturation Headways

The base saturation headways for each of the 512 combinations can be determined with Exhibit 21-15. They are adjusted by using the adjustment factors calculated in Step 4 and added to the base saturation headways to determine saturation headways.

For the example of eastbound Lane 1, these computations are shown in Rows 3118–3629 of the spreadsheet (Columns M–O).

Step 10: Compute Departure Headways

The departure headway of the lane is the sum of the products of the adjusted probabilities and the saturation headways. For the example of eastbound Lane 1, these computations are made by summing the product of Columns O and Y for Rows 3118–3629 in the example spreadsheet.

Step 11: Check for Convergence

The calculated values of h_d are checked against the assumed initial values for h_d. After one iteration, each calculated headway differs from the initial value by more than 0.1 s. Therefore, the new calculated headway values are used as initial values in a second iteration. For this problem, five iterations were required for convergence, as shown in Exhibit 32-24.

Exhibit 32-24
AWSC Example Problem 2: Convergence Check

	EB L1	EB L2	EB L3	WB L1	WB L2	WB L3	NB L1	NB L2	NB L3	SB L1	SB L2	SB L3
Total lane flow rate	56	216		156	164		76	164	116	48	124	88
hd, initial value, Iteration 1	3.2	3.2		3.2	3.2		3.2	3.2	3.2	3.2	3.2	3.2
x, initial, Iteration 1	0.0498	0.192		0.1387	0.1458		0.0676	0.1458	0.1031	0.0427	0.1102	0.0782
hd, computed value, Iteration 1	6.463	5.755		6.405	5.597		6.440	5.935	5.228	6.560	6.055	5.347
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 2	6.463	5.755		6.405	5.597		6.440	5.935	5.228	6.560	6.055	5.347
x, initial, Iteration 2	0.1005	0.3453		0.2776	0.255		0.136	0.2704	0.1685	0.0875	0.2086	0.1307
hd, computed value, Iteration 2	7.550	6.838		7.440	6.629		7.537	7.027	6.313	7.740	7.230	6.515
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 3	7.550	6.838		7.440	6.629		7.537	7.027	6.313	7.740	7.230	6.515
x, initial, Iteration 3	0.1174	0.4103		0.3224	0.302		0.1591	0.3201	0.2034	0.1032	0.249	0.1593
hd, computed value, Iteration 3	7.970	7.257		7.854	7.041		7.954	7.442	6.725	8.187	7.675	6.957
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 4	7.970	7.257		7.854	7.041		7.954	7.442	6.725	8.187	7.675	6.957
x, initial, Iteration 4	0.124	0.4354		0.3404	0.3208		0.1679	0.339	0.2167	0.1092	0.2643	0.17
hd, computed value, Iteration 4	8.130	7.416		8.010	7.196		8.114	7.601	6.884	8.359	7.845	7.126
Convergence?	N	N		N	N		N	N	N	N	N	N
hd, initial value, Iteration 5	8.130	7.416		8.010	7.196		8.114	7.601	6.884	8.359	7.845	7.126
x, initial, Iteration 5	0.1265	0.445		0.3471	0.3278		0.1713	0.3463	0.2218	0.1115	0.2702	0.1742
hd, computed value, Iteration 5	8.191	7.476		8.069	7.255		8.174	7.661	6.943	8.424	7.910	7.190
Convergence?	Y	Y		Y	Y		Y	Y	Y	Y	Y	Y

Step 12: Compute Capacity

As noted in the procedure, the capacity of each lane in a subject approach is computed by increasing the given flow rate on the subject lane (assuming the flows on the opposing and conflicting approaches are constant) until the degree of utilization for the subject lane reaches 1. This level of calculation requires running an iterative procedure many times, which is practical only for a spreadsheet or software implementation.

For this example, the capacity of eastbound Lane 1 can be found to be approximately 420 veh/h. This value is lower than the value that could be estimated by dividing the lane volume by the degree of utilization (56/0.1265 = 443 veh/h). The difference is due to the interaction effects among the approaches: increases in eastbound traffic volume increase the departure headways of the lanes on the other approaches, which increases the departure headways of the lanes on the subject approach.

Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time by using Equation 21-29. For the eastbound Lane 1 (using a value for m of 2.3 for Geometry Group 6), the calculation is as follows:

$$t_{s,EB,1} = h_{d,EB,1} - m = 8.19 - 2.3 = 5.89 \text{ s}$$

Step 14: Compute Control Delay and Determine LOS for Each Lane

The control delay for each lane is computed with Equation 21-30 as follows (eastbound Lane 1 illustrated):

$$d_{EB,1} = t_{s,EB,1} + 900T\left[\left(x_{EB,1} - 1\right) + \sqrt{(x_{EB,1} - 1)^2 + \frac{h_{d,EB,1}x_{EB,1}}{450T}}\right] + 5$$

$$d_{EB,1} = 5.89 + 900(0.25)\left[(0.1274 - 1) + \sqrt{(0.1274 - 1)^2 + \frac{8.19(0.1274)}{450(0.25)}}\right] + 5$$

$$d_{EB,1} = 12.1 \text{ s}$$

On the basis of Exhibit 20-2, eastbound Lane 1 is assigned LOS B.

Step 15: Compute Control Delay and Determine LOS for Each Approach and the Intersection

The control delay for each approach is calculated using Equation 21-31 as follows (eastbound approach illustrated):

$$d_{EB} = \frac{(12.1)(272) + (16.1)(216)}{56 + 216} = 15.3 \text{ s}$$

This value of delay is assigned LOS C.

Similarly, the control delay for the intersection is calculated as follows:

$$d_{\text{intersection}} = \frac{(15.3)(272) + (14.3)(320) + (13.1)(356) + (12.6)(260)}{272 + 320 + 356 + 260} = 14.0 \text{ s}$$

This value of delay is assigned LOS B.

Step 16: Compute Queue Lengths

The 95th percentile queue for each lane is computed with Equation 21-33 as follows for eastbound Lane 1:

$$Q_{95,EB1} \approx \frac{900(0.25)}{8.19}\left[(0.1274 - 1) + \sqrt{(0.1274 - 1)^2 + \frac{8.19(0.1274)}{150(0.25)}}\right]$$

$$Q_{95,EB1} \approx 0.4 \text{ veh}$$

This queue length commonly would be rounded up to one vehicle.

Discussion

The overall results can be found in the "DelayLOS" spreadsheet tab. As indicated in the output, all movements at the intersection are operating well with small delays. The worst-performing movement is eastbound Lane 2, which is operating with a volume-to-capacity ratio of 0.45 and a control delay of 16.1 s/veh, which results in LOS C. The intersection as a whole operates at LOS B, so the reporting of individual movements is important to avoid masking results caused by aggregating delays.