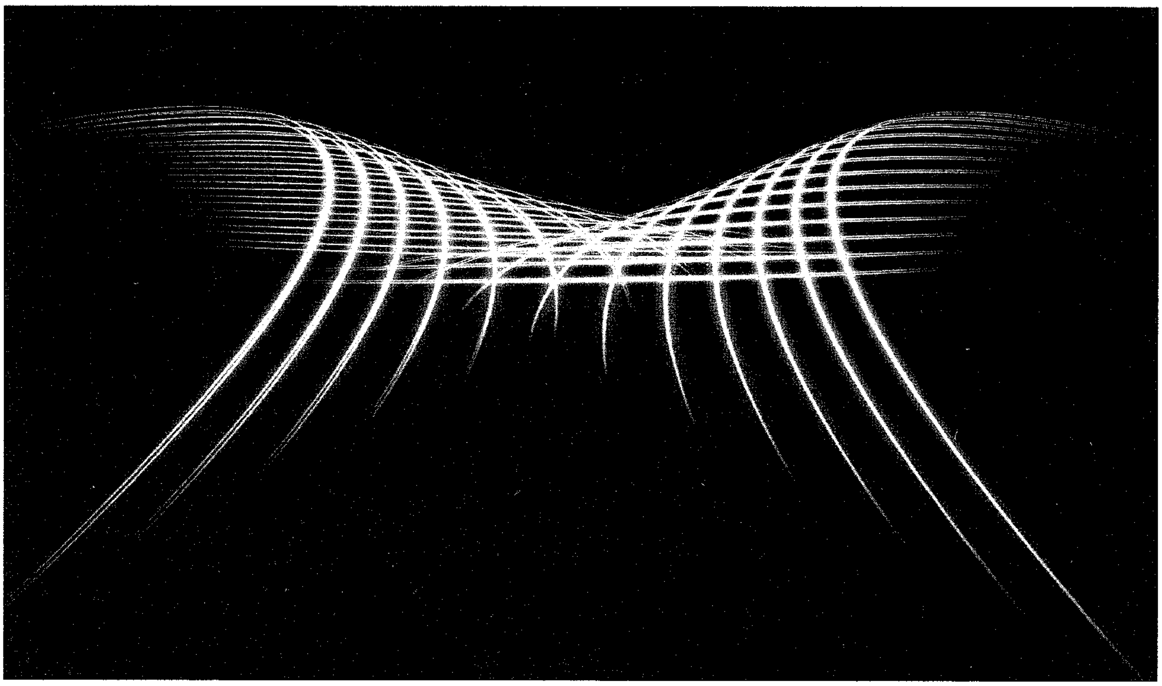
HCM2010

HIGHWAY CAPACITY MANUAL



VOLUME 3: INTERRUPTED FLOW

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VOLUME 3
INTERRUPTED FLOW

OVERVIEW

Volume 3 of the *Highway Capacity Manual* (HCM) contains eight chapters that present analysis methods for interrupted-flow system elements—that is, roadways and pathways that have fixed causes of periodic delay or interruption to the traffic stream, such as traffic signals and STOP signs. This volume addresses the following types of interrupted-flow system elements:

- *Urban street segments and facilities,* which are portions of roadways that have traffic signals, roundabouts, or STOP-controlled intersections spaced less than 2 mi apart on average;
- *Intersections,* consisting of signalized intersections, two-way STOP-controlled intersections, all-way STOP-controlled intersections, roundabouts, and interchange ramp terminals; and
- *Off-street pedestrian and bicycle facilities* that (*a*) are used only by nonmotorized modes and (*b*) are not considered part of an urban street or transit facility.

VOLUME ORGANIZATION

Urban Street Segments and Facilities

Urban streets typically serve multiple travel modes, in particular the automobile, pedestrian, bicycle, and transit modes. Travelers associated with each of these modes perceive the service provided to them by the urban street in different ways. Design or operational decisions that are intended to improve the service provided to one mode using an urban street can have both adverse and beneficial impacts on the service provided to other modes. The challenge for the analyst is to design and operate an urban street in such a way that all relevant travel modes are reasonably accommodated.

For the purpose of analysis, urban streets are separated into individual elements that are physically adjacent and operate as a single entity in serving travelers. A *point* represents the boundary between links and is represented by an intersection or ramp terminal. A *link* represents a length of roadway between two points. A link and its boundary points are referred to as a *segment.* Multiple contiguous segments can be combined into a single *facility.*

Chapter 17, Urban Street Segments, provides an integrated multimodal methodology for evaluating the quality of service provided to road users traveling along an urban street segment. **Chapter 16, Urban Street Facilities,** provides a similar methodology for evaluating extended lengths of urban streets. In both chapters, level of service (LOS) is reported separately for each mode and is not combined into a single overall LOS. This restriction recognizes that trip purpose, length, and expectation for each mode are different and that their combination does not produce a meaningful result. This integrated multimodal approach allows analysis of urban streets from a "complete streets" perspective.

Intersections

Five chapters in Volume 3 provide analysis methods for the different kinds of intersections that may be encountered along an urban street.

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

Chapter 19, Two-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing intersections where one street—the *major street*—is uncontrolled, while the other street—the *minor street*—is controlled by STOP signs. These intersections typically occur in one of two configurations: three-leg, where the single minor-street approach is controlled by a STOP sign, and four-leg, where both minor-street approaches are controlled by STOP signs. The methodology is applicable to major streets with up to six lanes, three in each direction. Chapter 19 also provides a methodology for estimating pedestrian delay and LOS in the crossing of major streets at two-way STOP-controlled intersections and at unsignalized midblock crossings.

Chapter 20, All-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing these types of intersections. All-way STOP-controlled intersections require every vehicle to stop at the intersection before proceeding. Because each driver must stop, the decision to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after stopping. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that no vehicles are currently in the intersection and that it is the driver's turn to proceed.

Chapter 21, Roundabouts, presents concepts and procedures for analyzing modern roundabouts. Roundabouts are intersections with a generally circular shape, characterized by yield on entry and circulation around a central island (counterclockwise in the United States). The methodology can be used to assess the operational performance of existing or planned one-lane or two-lane roundabouts.

Chapter 22, Interchange Ramp Terminals, addresses interchanges with signalized intersections, interchanges with roundabouts, and the impact and operations of adjacent closely spaced intersections. Interchange ramp terminals provide the connection between various highway facilities (e.g., freeway–arterial, arterial–arterial), and thus their efficient operation is essential. In addition, they need to provide adequate capacity to avoid affecting the connecting facilities. The chapter's methodology can be applied to the operational and planning level analysis of a broad range of interchange types, including diamond, partial cloverleaf, and single-point urban interchanges.

Off-Street Pedestrian and Bicycle Facilities

Chapter 23, Off-Street Pedestrian and Bicycle Facilities, provides capacity and LOS estimation procedures for the following types of facilities:

- *Walkways*: paved paths, ramps, and plazas that are generally located more than 35 ft from an urban street as well as streets reserved for pedestrian traffic on a full- or part-time basis;
- *Stairways*: staircases that are part of a longer pedestrian facility;
- *Shared-use paths*: paths physically separated from highway traffic for the use of pedestrians, bicyclists, runners, inline skaters, and other users of nonmotorized modes; and
- *Exclusive off-street bicycle paths*: paths physically separated from highway traffic for the exclusive use of bicycles.

On-street pedestrian and bicycle facilities are addressed in the other Volume 3 chapters, particularly Chapters 16–19.

RELATED CHAPTERS

Volume 1

The chapters in Volume 3 assume that the reader is already familiar with the concepts presented in the Volume 1 chapters, in particular the following:

- *Chapter 2, Applications*—types of HCM analysis, types of roadway system elements, and traffic flow characteristics;
- *Chapter 3, Modal Characteristics*—variations in demand, peak and analysis hours, *K*- and *D*-factors, facility types by mode, and interactions between modes;
- *Chapter 4, Traffic Flow and Capacity Concepts*—traffic flow parameters and factors that influence capacity; and
- *Chapter 5, Quality and Level-of-Service Concepts*—performance measures, service measures, and LOS.

Volume 2

Urban streets with two or more lanes in each direction that have traffic signals spaced 2 mi or more apart on average are treated as multilane highways. Chapter 14 provides analysis methods for two-lane highways. Two-lane roadways passing through moderately developed areas, such as small towns or developed recreational areas, are treated as Class III two-lane highways and can be analyzed with the methods given in Chapter 15.

Access Volume 4 at www.HCM2010.org

Volume 4

Seven chapters in Volume 4 (accessible at www.HCM2010.org) provide additional information that supplements the material presented in Volume 3. These chapters are as follows:

- *Chapter 29, Urban Street Facilities: Supplemental*—examples of applying alternative tools to situations not addressed by the Chapter 16 method for urban street facilities;
- *Chapter 30, Urban Street Segments: Supplemental*—methods for adjusting traffic demand to account for capacity constraints and midsegment turning movements, analyzing vehicular traffic flow on a segment bounded by signalized intersections, and estimating major-street delay due to midblock turns; a quick-estimation method for evaluating the operation of a coordinated street segment; a description of field measurement techniques; and documentation of the computational engine;
- *Chapter 31, Signalized Intersections: Supplemental*—descriptions of traffic signal concepts and field measurement techniques; details of procedures for calculating capacity, phase duration, delay, and back-of-queue; a quick-estimation method for determining an intersection's critical volume-to-capacity ratio, signal timing, and delay; documentation of the computational engine; and examples of applying alternative tools to situations not addressed by the Chapter 18 method for signalized intersections;
- *Chapter 32, STOP-Controlled Intersections: Supplemental*—methods for determining the potential capacity of two-way STOP-controlled intersections, the operation of two-way STOP-controlled intersections with pedestrian effects, and the operation of all-way STOP-controlled intersections with three-lane approaches; and additional example problems;
- *Chapter 33, Roundabouts: Supplemental*—guidance on lane-use assignment and calibrating the Chapter 21 capacity model;
- *Chapter 34, Interchange Ramp Terminals: Supplemental*—complete solutions to the Chapter 22 example problems, along with additional example problems; and
- *Chapter 35, Active Traffic Management*—descriptions of active traffic management strategies; a discussion of the mechanisms by which they affect demand, capacity, and performance; and general guidance on possible evaluation methods for active traffic management techniques.

The *HCM Applications Guide* in Volume 4 provides three case studies on the analysis of interrupted-flow facilities:

- *Case Study No. 1* illustrates the process of applying HCM techniques to questions relating to the operational and control needs of an intersection located along a roadway in a university town;

- *Case Study No. 2* illustrates the process of applying HCM techniques to the analysis of unsignalized intersections, signalized intersections, and urban streets in the context of a traffic impact analysis; and
- *Case Study No. 5* illustrates the consideration of nonautomobile modes as part of the evaluation of signalized and unsignalized intersections.

These case studies focus on the process of applying the HCM rather than on the details of performing calculations (which are addressed by the example problems in the Volume 3 and supplemental Volume 4 chapters). The case studies' computational results were developed by using HCM2000 methodologies and therefore may not match the results obtained from applying the HCM 2010. However, the process of application is the focus, not the specific computational results.

The Technical Reference Library in Volume 4 contains copies of (or links to) many of the documents referenced in Volume 3 and its supplemental chapters. The Technical Reference Library also provides computational engines (spreadsheets) to assist with the application of the urban streets, signalized intersections, all-way STOP-controlled intersections, and roundabouts methodologies.

CHAPTER 16
URBAN STREET FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 16, Urban Street Facilities, describes an integrated multimodal methodology for evaluating the quality of service provided to road users traveling along an urban street. An urban street is unique among road types because it typically serves multiple travel modes. Four of the more common urban street travel modes include automobile, pedestrian, bicycle, and transit. Travelers associated with each of these modes use different criteria to evaluate the service provided to them when they travel along an urban street. This integrated multimodal approach allows analysts to analyze urban streets from a "complete streets" perspective.

Design or operational decisions that are intended to improve the service provided to one mode can sometimes have an adverse impact on the service provided to another mode. The challenge for the analyst is to design and operate the urban street in such a way that all relevant travel modes are reasonably accommodated. The methodology described in this chapter is intended to assist the analyst in this regard by providing a means to assess the performance of each urban street travel mode.

OVERVIEW OF THE METHODOLOGY

This chapter's methodology is applicable to an urban or suburban street. The street is classified as an arterial or collector with one-way or two-way vehicular traffic flow. The intersections along the street can be signalized or unsignalized.

Analysis Level

Analysis level describes the level of detail used in applying the methodology. Three levels are recognized:

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

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

Study Period and Analysis Period

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

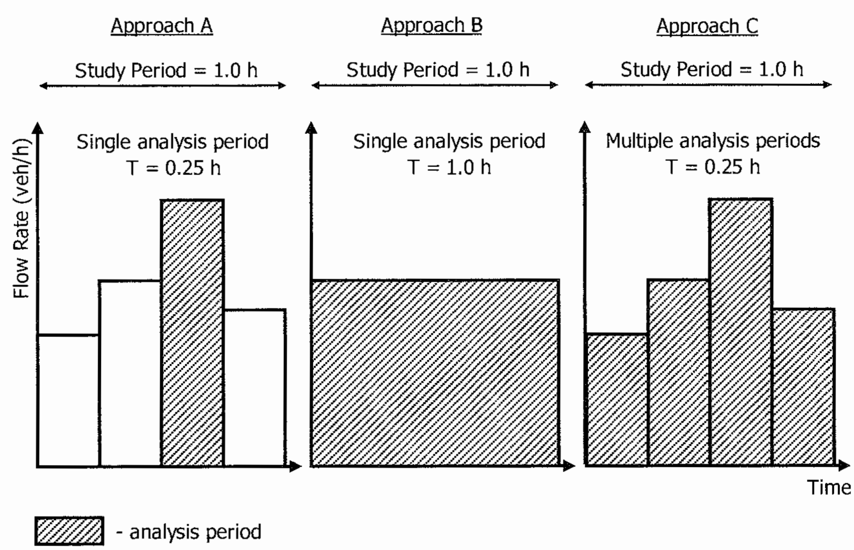
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the duration of the analysis period is in the range of 0.25 to 1 h. The longer durations in this range are sometimes used for planning 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.

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

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be 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.

Exhibit 16-1 demonstrates three alternative approaches an analyst might use for a given evaluation. Note that other alternatives exist and that the study period can exceed 1 h. Approach A is the one that has traditionally been used and, unless otherwise justified, is the one that is recommended for use.

Exhibit 16-1
Three Alternative Study Approaches



Approach A is based on the evaluation of the peak 15-min period during the study period. The analysis period, T, is 0.25 h. The equivalent hourly flow rate in vehicles per hour (veh/h) used for the analysis is based on either a peak 15-min traffic count multiplied by four or a 1-h demand volume divided by the peak hour factor. The former option is preferred whenever traffic counts are available. The peak hour factor equals the hourly count of vehicles divided by four times the peak 15-min count for a common hour interval. It is provided by the analyst or operating agency.

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. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay.

Performance Measures

An urban street's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road user group. Performance measures cited in this chapter include automobile travel speed, automobile stop rate, pedestrian space, pedestrian travel speed, pedestrian perception score, bicycle travel speed, bicycle perception score, transit travel speed, and transit passenger perception score.

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

Travel Modes

This chapter describes a separate methodology for evaluating urban street performance from the perspective of motorists, pedestrians, bicyclists, or transit passengers. These methodologies are referred to hereafter as the automobile methodology, pedestrian methodology, bicycle methodology, and transit methodology.

Each methodology consists of a set of procedures for computing the quality of service provided to one mode. Collectively, they can be used to evaluate urban street operation from a multimodal perspective.

Each methodology is focused on the evaluation of the urban street. A methodology for evaluating the segments that make up the street is described in Chapter 17, Urban Street Segments.

The four methodologies described in this chapter are based largely on the products of two National Cooperative Highway Research Program projects (*1, 2*).

The transit methodology described in this chapter is applicable to the evaluation of passenger service provided by local public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. Nonlocal transit vehicle speed and delay are evaluated by using the automobile methodology.

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

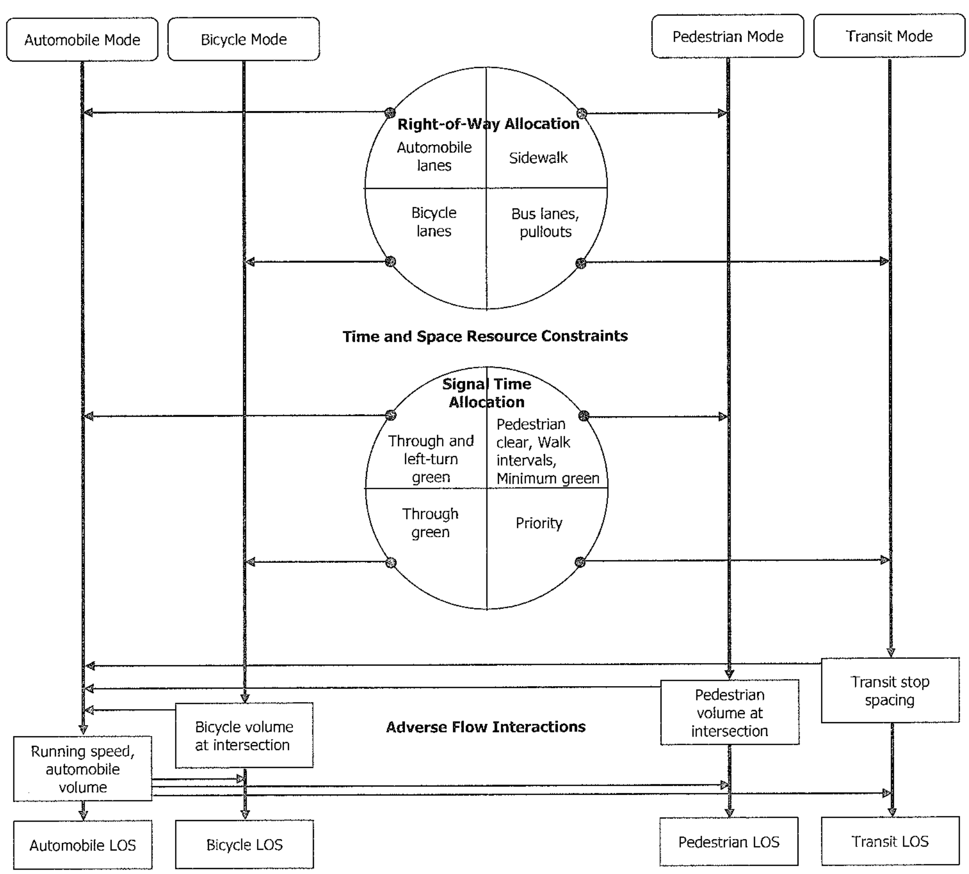
Multimodal Evaluation Framework

The urban street right-of-way is typically shared by multiple travel modes. Travelers associated with the more common modes include motorists, pedestrians, bicyclists, and transit passengers. The factors that influence the quality of service provided to these travelers vary by mode because each mode has a different trip purpose, length, and expectation.

The shared street right-of-way typically requires that the modes operate in close proximity to each other, sometimes even sharing the same portion of the cross section (e.g., a vehicular traffic lane). This arrangement may be workable when the modes are characterized by low demand volumes; however, acceptable operation for moderate to high volumes typically requires the spatial separation of the modes along the street and temporal (i.e., signal) separation at the intersections.

The integrated methodology described in Section 2 can be used to evaluate simultaneously the LOS provided to each travel mode on an urban street. A framework for this evaluation is shown in Exhibit 16-2.

Exhibit 16-2
Integrated Multimodal Evaluation Framework



The framework shown in Exhibit 16-2 illustrates the integrated multimodal evaluation approach supported by the methodology in Section 2. It is important to note that the LOS provided to each travel mode is separately evaluated. The relative importance given to each mode's LOS should be determined by the analyst (or operating agency) and reflect consideration of the subject street's functional class and purpose. The LOS for each mode should *not* be combined into one overall LOS for the street. This restriction recognizes that trip purpose, length, and expectation for each mode are different and that their combination does not produce a meaningful result.

Exhibit 16-2 illustrates how the travel modes compete for limited right-of-way along the street and at the intersections. They also compete for limited signal time at the intersections. For a given right-of-way, the allocation of space to one mode often requires a reduction (or elimination) of space for other modes and a corresponding reduction in their service quality.

The lower part of Exhibit 16-2 illustrates the potential adverse interactions between the automobile mode and the other modes. As the volume or speed of the automobile traffic stream increases, the LOS for the other modes may decrease. In contrast, if bicycle, pedestrian, or transit flows increase, then the LOS for the automobile traffic stream may decrease. In general, changes that alter resource allocation or flow interaction to improve the LOS for one mode may affect the other modes.

URBAN STREET FACILITY DEFINED

For the purpose of analysis, the urban street is separated into individual elements that are physically adjacent and operate as a single entity for the purpose of serving travelers. Two elements are commonly found on an urban street system: points and links. A *point* represents the boundary between links and is usually represented by an intersection or ramp terminal. A *link* represents a length of roadway between two points. A link and its boundary intersections are referred to as a *segment.* An urban street *facility* is a length of roadway that is composed of contiguous urban street segments and is typically functionally classified as an urban arterial or collector street.

Previous editions of this manual have allowed the evaluation of one direction of travel along a facility (even when it served two-way traffic). This approach is retained in this chapter 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 automobile methodology, a segment evaluation considers both directions of travel (when the street serves two-way traffic).

For the analysis of automobile 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 automobile methodology in this edition of the *Highway Capacity Manual* (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 these reasons, it is important to evaluate both travel directions on a two-way segment.

Facility Length Considerations

Urban arterial and collector streets are designed to accommodate longer trips than local streets. They also have a significant mobility function and support the hierarchy of movement by connecting to streets of higher and lower functional class. An urban street facility with these attributes typically has a length of 1 mi or more in downtown areas and 2 mi or more in other areas. When an urban street facility meets or exceeds this length, average travel speed is a more meaningful indication of facility performance and LOS.

At least one intersection (or ramp terminal) along the facility must have a type of control that can impose on the through movement a legal requirement to stop or yield. A significant change in one or more facility characteristics may indicate the end of one facility and the start of a second facility. These characteristics include cross section features (e.g., number of through lanes, shoulder width, curb presence), annual average daily traffic volume, roadside development density and type, and vehicle speed. One or more of these characteristics will often change significantly when the street crosses an urban-to-suburban area boundary or intersects a freeway interchange.

If a facility assessment is desired for a given travel mode, the analyst will need to evaluate all of the segments that make up the facility for a common travel direction and aggregate the performance measures for each segment to obtain a facility performance estimate.

Facility Versus Segment Analysis Scope

The methodology described in Section 2 is used to evaluate an entire facility; however, for some specific conditions it may not be necessary to evaluate the entire facility. For these conditions, the appropriate segment or intersection chapter methodology may be used alone to evaluate selected segments or intersections. In general, it is up to the analyst to determine the scope of each analysis (i.e., one intersection, one segment, two segments, or all segments on the facility) on the basis of analysis objectives and agency directives.

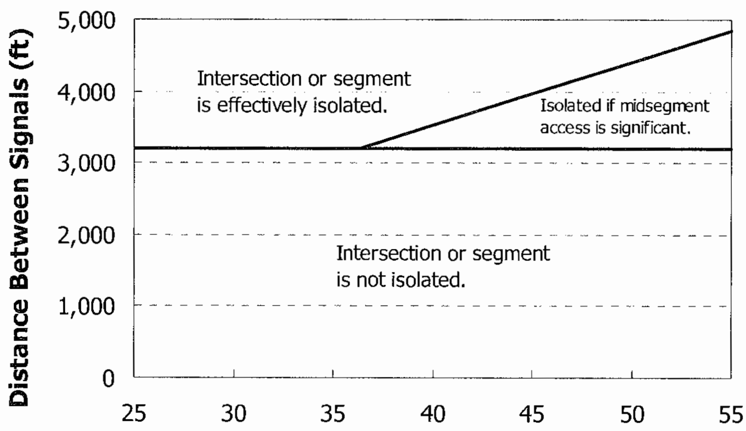
One condition for which it may be acceptable to evaluate an individual segment or intersection occurs when the segment or intersection is considered to operate in *isolation* from upstream signalized intersections. A segment or intersection that is effectively isolated experiences negligible influence from upstream signalized intersections. Flow on an isolated segment or at an isolated intersection is effectively random over the cycle and without a discernible platoon pattern evident in the cyclic profile of arrivals. These characteristics are more likely to be found when (*a*) the nearest upstream signalized intersection is sufficiently distant from the subject segment or intersection and (*b*) the subject segment or intersection, if signalized, is not coordinated with the upstream signal.

A segment or intersection is sufficiently distant from the nearest upstream signal if an intermediate intersection uses stop or yield control to regulate through traffic on the facility. If there is no intermediate STOP- or YIELD-controlled intersection, then Exhibit 16-3 can be used to obtain an indication of whether a segment or intersection is sufficiently distant from an upstream signal. If the distance between signals is above the trend line, then the subject intersection or segment is likely to operate as effectively isolated (provided that it is not coordinated with the upstream signal).

LOS CRITERIA

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

Exhibit 16-3
Signal Spacing Associated with Effectively Isolated Operation



Automobile Mode

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

Through-vehicle travel speed is used to characterize vehicular LOS for a given direction of travel along an urban street facility. This speed reflects the factors that influence running time along each link and the delay incurred by through vehicles at each boundary intersection. This performance measure indicates the degree of mobility provided by the facility. The following paragraphs characterize each service level.

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 intersections is minimal. The travel speed exceeds 85% of the base free-flow speed.

LOS B describes reasonably unimpeded operation. The ability to maneuver within the traffic stream is only slightly restricted and control delay at the boundary intersections is not significant. The travel speed is between 67% and 85% of the base free-flow speed.

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 intersections may contribute to lower travel speeds. The travel speed is between 50% and 67% of the base free-flow speed.

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 intersections. The travel speed is between 40% and 50% of the base free-flow speed.

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 intersections. The travel speed is between 30% and 40% of the base free-flow speed.

LOS F is characterized by flow at extremely low speed. Congestion is likely occurring at the boundary intersections, as indicated by high delay and extensive queuing. The travel speed is 30% or less of the base free-flow speed. Also, LOS F is assigned to the subject direction of travel if the through movement at one or more boundary intersections has a volume-to-capacity ratio greater than 1.0.

Exhibit 16-4 lists the LOS thresholds established for the automobile mode on urban streets.

Exhibit 16-4
LOS Criteria: Automobile Mode

Travel Speed as a Percentage of Base Free-Flow Speed (%)	LOS by Critical Volume-to-Capacity Ratio[a]	
	≤ 1.0	> 1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

Note: [a] The critical volume-to-capacity ratio is based on consideration of the through movement volume-to-capacity ratio at each boundary intersection in the subject direction of travel. The critical volume-to-capacity ratio is the largest ratio of those considered.

Nonautomobile 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 in Chapter 17, Urban Street Segments, and Chapter 18, Signalized Intersections, provide procedures for mathematically combining these factors into a score for the segment or intersection, respectively. This score is then used in this chapter to determine the LOS that is provided for a given direction of travel along a facility.

Exhibit 16-5 lists the range of scores associated with each LOS for the pedestrian travel mode. The LOS for this particular mode 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.

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 respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

Exhibit 16-5
LOS Criteria: Pedestrian Mode

Pedestrian LOS Score	LOS by Average Pedestrian Space (ft^2/p)					
	>60	>40–60	>24–40	>15–24	>8.0–15[a]	≤ 8.0[a]
≤2.00	A	B	C	D	E	F
>2.00–2.75	B	B	C	D	E	F
>2.75–3.50	C	C	C	D	E	F
>3.50–4.25	D	D	D	D	E	F
>4.25–5.00	E	E	E	E	E	F
>5.00	F	F	F	F	F	F

Note: [a] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Exhibit 16-6 lists the range of scores that are associated with each LOS for the bicycle and transit modes. This exhibit is also applicable for determining pedestrian LOS when a sidewalk is not available.

Exhibit 16-6
LOS Criteria: Bicycle and Transit Modes

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

REQUIRED INPUT DATA

This subsection describes the required input data for the automobile, pedestrian, bicycle, and transit methodologies.

Automobile Mode

This part describes the input data needed for the automobile methodology. The data are listed in Exhibit 16-7 and are identified as "input data elements." For the subject travel direction, these elements must be provided for each segment and for the through-movement group at each boundary intersection.

The last column in Exhibit 16-7 indicates 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 segment are identified in Chapter 17, Urban Street Segments. Similarly, the input data needed to evaluate the boundary intersections are identified in the appropriate chapter (i.e., Chapters 18 to 22).

Exhibit 16-7
Input Data Requirements: Automobile Mode

Data Category	Location	Input Data Element	Basis
Geometric Design	Segment	Segment length	Segment
Other	Segment	Analysis period duration	Facility
Performance Measures	Boundary intersection	Volume-to-capacity ratio	Through-movement group
	Segment	Base free-flow speed	Segment
		Travel speed	Segment

Notes: Through-movement group = one value for the segment through movement at the downstream boundary intersection (inclusive of any turn movements in a shared lane).
Segment = one value or condition for each segment and direction of travel on the facility.
Facility = one value or condition for the facility.

Segment Length

Segment length represents 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, then an average length is used. One length is needed for each segment on the facility.

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. Also, if the analysis period is other than 15 min, then the peak hour factor should not be used.

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

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

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

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

If a multiple-period analysis is used and the boundary intersections are signalized, then the procedure described in Chapter 18 should be used to guide the evaluation. When a procedure for multiple-period analysis is not provided in the chapter that corresponds to the boundary intersection configuration, the analyst should separately evaluate each period and use the residual queue from one period as the initial queue for the next period.

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

Volume-to-Capacity Ratio

This volume-to-capacity ratio is for the lane group serving the through movement that exits the segment at the downstream boundary intersection. With one exception, a procedure for computing this ratio is described in the appropriate intersection chapter (i.e., Chapters 18 to 22). Chapter 19, 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 16-1:

Equation 16-1

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

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 19-43 in Chapter 19. It is equal to 1.0 if a left-turn bay is provided for left turns from the major street.

One volume-to-capacity ratio is needed for the downstream boundary intersection of each segment on the facility.

Base Free-Flow Speed

The base free-flow speed characterizes the traffic speed on the segment when free-flow conditions exist and speed is uninfluenced by signal spacing. A procedure for determining this speed is described in Chapter 17. One speed is needed for each travel direction on each segment on the facility.

Travel Speed

Travel speed represents the ratio of segment length to through-movement travel time. Travel time is computed as the sum of segment running time and through-movement control delay at the downstream boundary intersection. A procedure for computing travel speed is described in Chapter 17. One speed is needed for each travel direction of each segment on the facility.

Nonautomobile Modes

This part describes the input data needed for the pedestrian, bicycle, and transit methodologies. The data are listed in Exhibit 16-8 and are identified as "input data elements." They must be separately specified for each direction of travel on the facility. Segment length is defined in the previous part.

Exhibit 16-8 categorizes each input data element by travel mode methodology. An "X" is used to indicate the association between a data element and methodology. A blank cell indicates that the data element is not used as input for the corresponding methodology.

Exhibit 16-8
Input Data Requirements: Nonautomobile Modes

Data Category	Location	Input Data Element	Pedestrian Mode	Bicycle Mode	Transit Mode
Geometric Design	Segment	Segment length	X	X	X
		Presence of a sidewalk	X		
Performance Measures	Segment	Pedestrian space	X		
		Pedestrian travel speed	X		
		Pedestrian LOS score for segment	X		
		Bicycle travel speed		X	
		Bicycle LOS score for segment		X	
		Transit travel speed			X
		Transit LOS score for segment			X

Presence of a Sidewalk

A sidewalk is a paved walkway that is provided at the side of the roadway. It is assumed that pedestrians will walk in the street if a sidewalk is not present. An indication of sidewalk presence is needed for each side of interest for each segment on the facility.

Pedestrian Space

Pedestrian space is a performance measure that describes the average circulation area available to each pedestrian traveling along the sidewalk. A procedure is described in Chapter 17 for estimating this quantity for a given sidewalk. One value is needed for each sidewalk of interest associated with each segment on the facility.

Pedestrian Travel Speed

Pedestrian travel speed represents the ratio of segment length to pedestrian travel time. Travel time is computed as the sum of segment walking time and control delay at the downstream boundary intersection. A procedure for computing this travel speed is described in Chapter 17. One speed is needed for each sidewalk of interest associated with each segment on the facility.

Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is used in the pedestrian methodology to determine facility LOS. It is obtained from the pedestrian methodology in Chapter 17. One score is needed for each direction of travel of interest for each segment on the facility.

Bicycle Travel Speed

Bicycle travel speed represents the ratio of segment length to bicycle travel time. Travel time is computed as the sum of segment running time and control delay at the downstream boundary intersection. This speed is computed only when a bicycle lane is present on the segment. A procedure for computing this travel speed is described in Chapter 17. One speed is needed for each direction of travel of interest for each segment on the facility.

Bicycle LOS Score for Segment

The bicycle LOS score for the segment is used in the bicycle methodology to estimate facility LOS. It is obtained from the bicycle methodology in Chapter 17. One score is needed for each direction of travel of interest for each segment on the facility.

Transit Travel Speed

Transit travel speed represents the ratio of segment length to transit travel time. Travel time is computed as the sum of segment running time and control delay at the downstream boundary intersection. A procedure for computing this travel speed is described in Chapter 17. One speed is needed for each direction of travel of interest for each segment on the facility.

Transit LOS Score for Segment

The transit LOS score for the segment is used in the transit methodology to estimate facility LOS. It is obtained from the transit methodology in Chapter 17. One score is needed for each direction of travel of interest for each segment on the facility.

SCOPE OF THE METHODOLOGY

Four methodologies are presented in this chapter. One methodology is provided for each of the automobile, pedestrian, bicycle, and transit modes. This section identifies the conditions for which each methodology is applicable.

- **Signalized and two-way STOP-controlled boundary intersections.** All methodologies can be used to evaluate facility performance with signalized or two-way STOP-controlled boundary intersections. In the latter case, the cross street is STOP controlled. The automobile methodology can also be used to evaluate performance with all-way STOP- or YIELD-controlled (e.g., roundabout) boundary intersections.
- **Arterial and collector streets.** The four methodologies were developed with a focus on arterial and collector street conditions. If a methodology is used to evaluate a local street, then the performance estimates should be carefully reviewed for accuracy.
- **Steady flow conditions.** The four 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 four methodologies were developed to estimate the LOS perceived by automobile 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.
- **Target travel modes.** The automobile methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The pedestrian, bicycle, and transit methodologies address travel by walking, bicycle, and transit vehicle, respectively. The transit methodology is limited to the evaluation of public transit vehicles operating in mixed or exclusive traffic lanes and stopping along the street. The methodologies are not designed to evaluate the performance of other travel means (e.g., grade-separated rail transit, golf carts, or motorized bicycles).
- **Influences in the right-of-way.** A road user's perception of quality of service is influenced by many factors inside and outside 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, or 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.
- **Mobility focus for automobile methodology.** The automobile methodology is intended to facilitate the evaluation of mobility. Accessibility to adjacent properties by way of automobile is not directly evaluated with this methodology. Regardless, a segment's accessibility should also be considered when its performance is evaluated, especially if the street is intended to provide such access. Oftentimes, factors that favor mobility reflect minimal levels of access and vice versa.
- **"Typical pedestrian" focus for pedestrian methodology.** The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. As such, the performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to 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.

LIMITATIONS OF THE METHODOLOGY

The urban street facility methodology uses the performance measures estimated by the segment and intersection methodologies in Chapters 17 to 22. As such, it incorporates the limitations of these methodologies (which are identified in the respective segment or intersection chapter).

2. METHODOLOGY

OVERVIEW

This section describes four methodologies for evaluating the performance of an urban street facility. Each methodology addresses one possible travel mode within the street right-of-way. Analysts should choose the combination of methodologies that are appropriate for their analysis needs.

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

The focus of each methodology in this chapter is the facility. Methodologies for quantifying the performance of a segment or boundary intersection are described in other chapters (i.e., Chapters 17 to 22).

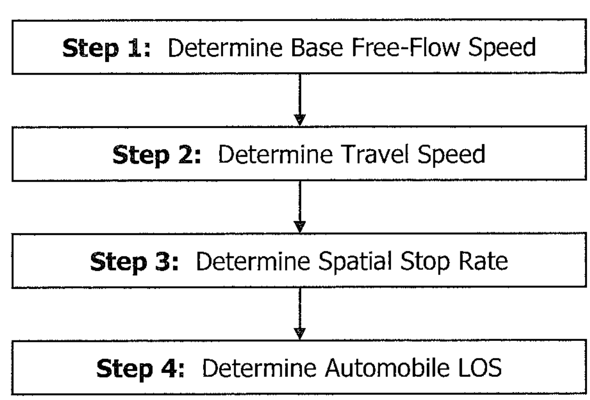
AUTOMOBILE MODE

This subsection provides an overview of the methodology for evaluating urban street facility performance from the motorist perspective. Each travel direction along the facility is separately evaluated. *Unless otherwise stated, all variables are specific to the subject direction of travel.*

The methodology is focused on the analysis of facilities with signalized, two-way STOP, all-way STOP, or roundabout boundary intersections. The signalized intersection can be an interchange ramp terminal.

Exhibit 16-9 illustrates the calculation framework of the automobile methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. The calculations are described more fully in the remainder of this subsection.

Exhibit 16-9
Automobile Methodology for Urban Street Facilities



Step 1: Determine Base Free-Flow Speed

The base free-flow speed for the facility is the basis for LOS determination. It is determined for each segment by using the procedures described in Chapter 17, Urban Street Segments. The base free-flow speed for the facility is calculated by using Equation 16-2:

$$S_{fo,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{fo,i}}}$$

Equation 16-2

where

$S_{fo,F}$ = base free-flow speed for the facility (mi/h),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$S_{fo,i}$ = base free-flow speed for segment i (mi/h).

Step 2: Determine Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed for the through-vehicle traffic stream that reflects the running speed along the street for through vehicles and any delay they may incur at the boundary intersections. The travel speed for through vehicles is determined for each segment by using the procedures described in Chapter 17. The travel speed for the facility is calculated by using Equation 16-3:

$$S_{T,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{T,seg,i}}}$$

Equation 16-3

where $S_{T,F}$ is the travel speed for the facility (mi/h), $S_{T,seg,i}$ is the travel speed of through vehicles for segment i (mi/h), and other variables are as previously defined.

Step 3: Determine Spatial Stop Rate

The spatial stop rate for the facility is the ratio of stop count to facility length. It relates the number of full stops incurred by the average through vehicle to the distance traveled. The spatial stop rate for through vehicles is determined for each segment by using the procedures described in Chapter 17. The spatial stop rate for the facility is calculated by using Equation 16-4:

Equation 16-4

$$H_F = \frac{\sum_{i=1}^{m} H_{seg,i}\ L_i}{\sum_{i=1}^{m} L_i}$$

where H_F is the spatial stop rate for the facility (stops/mi), $H_{seg,i}$ is the spatial stop rate for segment i (stops/mi), and other variables are as previously defined.

The spatial stop rate from Equation 16-4 can be used to estimate an automobile traveler perception score for the facility if desired. The equations in Step 10 of Chapter 17 are used for this purpose. The value of H_F would be substituted for H_{seg} in each equation. Similarly, the proportion of intersections with a left-turn lane P_{LTL} would be calculated for the entire facility and this one value used in each equation.

Step 4: Determine Automobile LOS

LOS is determined for both directions of travel along the facility. Exhibit 16-4 lists the LOS thresholds established for this purpose. As indicated in this exhibit, LOS is defined by travel speed, expressed as a percentage of the base free-flow speed. The base free-flow speed is computed in Step 1 and the travel speed is computed in Step 2.

The footnote to Exhibit 16-4 indicates that volume-to-capacity ratio for the through movement at the downstream boundary intersections is also relevant to the determination of facility LOS. This footnote indicates that LOS F is assigned to the subject direction of travel if a volume-to-capacity ratio greater than 1.0 exists for the through movement at one or more boundary intersections.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

PEDESTRIAN MODE

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to pedestrians.

Urban street facility 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.*

The methodology is focused on the analysis of facilities with either signal-controlled or two-way STOP-controlled boundary intersections. This edition of the HCM does not include a procedure for evaluating a facility's performance when a boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

The pedestrian methodology is applied through a series of four steps that culminate in the determination of the facility LOS. These steps are illustrated in Exhibit 16-10.

Exhibit 16-10
Pedestrian Methodology for Urban Street Facilities

Step 1: Determine Pedestrian Space

Step 2: Determine Pedestrian Travel Speed

Step 3: Determine Pedestrian LOS Score

Step 4: Determine Pedestrian LOS

Concepts

The methodology provides a variety of measures for evaluating facility performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip along the facility. One measure is the LOS score. This score is an indication of the typical pedestrian's perception of the overall facility travel experience. A second measure is the average speed of pedestrians traveling along the facility.

A third measure is based on the concept of "circulation area." It represents the average amount of sidewalk area available to each pedestrian walking along the facility. A larger area is more desirable from the pedestrian perspective. Exhibit 16-11 provides a qualitative description of pedestrian space that can be used to evaluate sidewalk performance from a circulation-area perspective.

Exhibit 16-11
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p) Random Flow	Pedestrian Space (ft^2/p) 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 16-11 indicate a sensitivity to flow condition. Random pedestrian flow is typical of most facilities. Platoon flow is appropriate for facilities made up of shorter segments (e.g., in downtown areas) with signalized boundary intersections.

Step 1: Determine 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 facility performance for travel in a sidewalk. This step is applicable only when the sidewalk exists on the subject side of the street.

The pedestrian space is determined for each segment by using the procedures described in Chapter 17, Urban Street Segments. The pedestrian space for the facility is calculated by using Equation 16-5:

Equation 16-5

$$A_{p,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{A_{p,i}}}$$

where

$A_{p,F}$ = pedestrian space for the facility (ft²/p),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$A_{p,i}$ = pedestrian space for segment i (ft²/p).

The pedestrian space for the facility reflects the space provided on the sidewalk along the segment. It does not consider the corner circulation area or the crosswalk circulation area at the intersections. Regardless, the analyst should verify that the intersection corners and crosswalks adequately accommodate pedestrians.

Step 2: Determine Pedestrian Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed of pedestrians that reflects their walking speed along the sidewalk and any delay they may incur at the boundary intersections. The travel speed for pedestrians is determined for each segment by using the procedures described in Chapter 17. The pedestrian travel speed for the facility is calculated by using Equation 16-6:

Equation 16-6

$$S_{Tp,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{Tp,seg,i}}}$$

where $S_{Tp,F}$ is the travel speed of through pedestrians for the facility (ft/s), $S_{Tp,seg,i}$ is the travel speed of through pedestrians for segment i (ft/s), and other variables are as previously defined.

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 3: Determine Pedestrian LOS Score

The pedestrian LOS score for the facility is computed in this step. It represents a length-weighted average of the pedestrian LOS scores for the individual segments that make up the facility. The segment scores are determined by using the procedures described in Chapter 17. The score for the facility is calculated by using Equation 16-7:

$$I_{p,F} = \frac{\sum_{i=1}^{m} I_{p,seg,i}\, L_i}{\sum_{i=1}^{m} L_i}$$

Equation 16-7

where

$I_{p,F}$ = pedestrian LOS score for the facility, and

$I_{p,seg,i}$ = pedestrian LOS score for segment *i*.

Other variables are as previously defined.

Step 4: Determine Pedestrian LOS

The pedestrian LOS for the facility is determined by using the pedestrian LOS score from Step 3 and the average pedestrian space from Step 1. These two performance measures are compared with their respective thresholds in Exhibit 16-5 to determine the LOS for the specified direction of travel along the subject facility. If the sidewalk does not exist and pedestrians are relegated to walking in the street, then LOS is determined by using Exhibit 16-6 because the pedestrian space concept does not apply.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

BICYCLE MODE

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to bicyclists.

Urban street facility 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 methodology is focused on the analysis of a facility with either signal-controlled or two-way STOP-controlled boundary intersections. This edition of the HCM does not include a procedure for evaluating a facility's performance when a boundary intersection is an all-way STOP-controlled intersection, a roundabout, or a signalized interchange ramp terminal.

The bicycle methodology is applied through a series of three steps that culminate in the determination of the facility LOS. These steps are illustrated in Exhibit 16-12.

Exhibit 16-12
Bicycle Methodology for Urban Street Facilities

Step 1: Determine Bicycle Travel Speed
↓
Step 2: Determine Bicycle LOS Score
↓
Step 3: Determine Bicycle LOS

Step 1: Determine Bicycle Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed of bicycles that reflects their running speed along the street and any delay they may incur at the boundary intersections. The travel speed for bicycles is determined for each segment by using the procedures described in Chapter 17. The bicycle travel speed for the facility is calculated by using Equation 16-8:

Equation 16-8

$$S_{Tb,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{Tb,seg,i}}}$$

where

$S_{Tb,F}$ = travel speed of through bicycles for the facility (mi/h),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$S_{Tb,seg,i}$ = travel speed of through bicycles for segment i (mi/h).

Step 2: Determine Bicycle LOS Score

The bicycle LOS score for the facility is computed in this step. It represents a length-weighted average of the bicycle LOS scores for the individual segments that make up the facility. The segment scores are determined by using the procedures described in Chapter 17. The score for the facility is calculated by using Equation 16-9:

Equation 16-9

$$I_{b,F} = \frac{\sum_{i=1}^{m} I_{b,seg,i}\ L_i}{\sum_{i=1}^{m} L_i}$$

where $I_{b,F}$ is the bicycle LOS score for the facility, $I_{b,seg,i}$ is the bicycle LOS score for segment i, and other variables are as previously defined.

Step 3: Determine Bicycle LOS

The bicycle LOS for the facility is determined by using the bicycle LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 16-6 to determine the LOS for the specified direction of travel along the subject facility.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

TRANSIT MODE

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to transit passengers.

Urban street facility 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.*

The methodology is applicable to public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. 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* (3).

The transit methodology is applied through a series of three steps that culminate in the determination of facility LOS. These steps are illustrated in Exhibit 16-13. If multiple routes exist on the segment, then each route is evaluated by using a separate application of this methodology.

Exhibit 16-13
Transit Methodology for Urban Street Facilities

Step 1: Determine Transit Travel Speed

↓

Step 2: Determine Transit LOS Score

↓

Step 3: Determine Transit LOS

Step 1: Determine Transit Travel Speed

The travel speed for the facility is the ratio of facility length to facility travel time. It represents an equivalent average speed of transit vehicles that reflects their running speed along the street and any delay they may incur at the boundary intersection. The travel speed for a transit vehicle is determined for each segment by using the procedures described in Chapter 17. The transit travel speed for the facility is calculated by using Equation 16-10:

Equation 16-10

$$S_{Tt,F} = \frac{\sum_{i=1}^{m} L_i}{\sum_{i=1}^{m} \frac{L_i}{S_{Tt,seg,i}}}$$

where

$S_{Tt,F}$ = travel speed of transit vehicles for the facility (mi/h),

L_i = length of segment i (ft),

m = number of segments on the facility, and

$S_{Tt,seg,i}$ = travel speed of transit vehicles for segment i (mi/h).

Step 2: Determine Transit LOS Score

The transit LOS score for the facility is computed in this step. It represents a length-weighted average of the transit LOS score for the individual segments that make up the facility. The segment scores are determined by using the procedures described in Chapter 17. The score for the facility is calculated by using Equation 16-11:

Equation 16-11

$$I_{t,F} = \frac{\sum_{i=1}^{m} I_{t,seg,i}\, L_i}{\sum_{i=1}^{m} L_i}$$

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

Step 3: Determine Transit LOS

The transit LOS for the facility is determined by using the transit LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 16-6 to determine the LOS for the specified direction of travel along the subject facility.

Facility LOS must be interpreted with caution. It can suggest acceptable operation of the facility when, in reality, certain segments are operating at an unacceptable LOS. For each travel direction, the analyst should always verify that each segment is providing acceptable operation and consider reporting the LOS for the poorest-performing segment as a means of providing context for the interpretation of facility LOS.

3. APPLICATIONS

TYPES OF ANALYSIS

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

Operational Analysis

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

Design Analysis

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

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

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.

Planning and Preliminary Engineering Analysis

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

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose are provided in Chapters 17 to 22.

USE OF ALTERNATIVE TOOLS

Chapter 29, Urban Street Facilities: Supplemental, includes a set of examples to illustrate the use of alternative tools to address the stated limitations of this chapter and Chapter 17, Urban Street Segments. Specifically, these examples are used to illustrate (*a*) the application of deterministic tools to optimize the signal timing, (*b*) the effect of using a roundabout as a segment boundary, (*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.

GENERALIZED DAILY SERVICE VOLUMES FOR URBAN STREET FACILITIES

Generalized daily service volume tables provide a means to assess a large number of urban streets in a region or jurisdiction quickly to determine which facilities need to be assessed more carefully (by using operational analysis) to ameliorate existing or pending problems.

To build a generalized daily service volume table for urban street facilities, a number of simplifying assumptions must be made. The assumptions made here include the following:

- All segments of the facility have the same number of through lanes (one, two, or three) in each direction;
- Only traffic signal control is used along the facility (i.e., no roundabouts or all-way STOP-controlled intersections exist);
- The traffic signals are coordinated and semi-actuated, the arrival type is 4, the traffic signal cycle time C is 120 s, and the weighted average green-to-cycle-length (g/C) ratio for through movements (defined below) is 0.45;
- Exclusive left-turn lanes with protected left-turn phasing and adequate queue storage are provided at each signalized intersection, and no exclusive right-turn lanes are provided;
- At each traffic signal, 10% of the traffic on the urban street facility turns left and 10% turns right;
- The peak hour factor is 0.92;
- The facility length is 2 mi, and no restrictive medians exist along the facility; and
- The base saturation flow rate s_o is 1,900 passenger cars per hour per lane (pc/h/ln).

The weighted average g/C ratio of an urban street is the average of the critical intersection through g/C ratio and the average of all the other g/C ratios for the urban street. For example, if there are four signals with a through g/C ratio of 0.50 and one signal with a through g/C ratio of 0.40, the weighted average g/C ratio for the urban street is 0.45. The weighted g/C ratio takes into account the adverse effect of the critical intersection and the overall quality of flow for the urban street.

Generalized daily service volumes are provided in Exhibit 16-14 for urban street facilities with posted speeds of 30 and 45 mi/h; two, four, or six lanes (both directions); and six combinations of the K-factor and D-factor. To use this table, analysts must select a combination of K and D appropriate for their locality.

The 30-mi/h values further assume an average traffic signal spacing of 1,050 ft and 20 access points/mi, while the 45-mi/h values assume an average traffic signal spacing of 1,500 ft and 10 access points/mi.

Exhibit 16-14
Generalized Daily Service Volumes for Urban Street Facilities (1,000 veh/day)

K-Factor	*D*-Factor	Two-Lane Streets				Four-Lane Streets				Six-Lane Streets			
		LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
Posted Speed = 30 mi/h													
0.09	0.55	NA	5.9	15.4	19.9	NA	11.3	31.4	37.9	NA	16.3	46.4	54.3
	0.60	NA	5.4	14.1	18.3	NA	10.3	28.8	34.8	NA	15.0	42.5	49.8
0.10	0.55	NA	5.3	13.8	17.9	NA	10.1	28.2	34.1	NA	14.7	41.8	48.9
	0.60	NA	4.8	12.7	16.4	NA	9.3	25.9	31.3	NA	13.5	38.3	44.8
0.11	0.55	NA	4.8	12.6	16.3	NA	9.2	25.7	31.0	NA	13.4	38.0	44.5
	0.60	NA	4.4	11.5	14.9	NA	8.4	23.5	28.4	NA	12.2	34.8	40.8
Posted Speed = 45 mi/h													
0.09	0.55	NA	10.3	18.6	19.9	NA	21.4	37.2	37.9	NA	31.9	54.0	54.3
	0.60	NA	9.4	17.1	18.3	NA	19.6	34.1	34.8	NA	29.2	49.5	49.8
0.10	0.55	NA	9.3	16.8	17.9	NA	19.3	33.5	34.1	NA	28.7	48.6	48.9
	0.60	NA	8.5	15.4	16.4	NA	17.7	30.7	31.3	NA	26.3	44.5	44.8
0.11	0.55	NA	8.4	15.3	16.3	NA	17.5	30.5	31.0	NA	26.1	44.2	44.4
	0.60	NA	7.7	14.0	14.9	NA	16.1	27.9	28.4	NA	23.9	40.5	40.7

Notes: NA = not applicable; LOS cannot be achieved with the stated assumptions.
General assumptions include no roundabouts or all-way STOP-controlled intersections along the facility; coordinated, semi-actuated traffic signals; arrival type 4; 120-s cycle time; protected left-turn phases; 0.45 weighted average *g/C* ratio; exclusive left-turn lanes with adequate queue storage provided at traffic signals; no exclusive right-turn lanes provided; no restrictive median; 2-mi facility length; 10% of traffic turns left and 10% turns right at each traffic signal; peak hour factor = 0.92; and base saturation flow rate = 1,900 pc/h/ln.
Additional assumptions for 30-mi/h facilities: signal spacing = 1,050 ft and 20 access points/mi.
Additional assumptions for 45-mi/h facilities: signal spacing = 1,500 ft and 10 access points/mi.

Exhibit 16-14 is provided for general planning use and should *not* be used to analyze any specific urban street facility or to make final decisions on important design features. A full operational analysis using this chapter's methodology is required for such specific applications.

The exhibit is useful, however, in evaluating the overall performance of a large number of urban streets within a jurisdiction, as a first pass to determine where problems might exist or arise, or to determine where improvements might be needed. Any urban street identified as likely to experience problems or need improvement, however, should then be subjected to a full operational analysis before any decisions on implementing specific improvements are made.

Daily service volumes are strongly affected by the *K*- and *D*-factors chosen as typical for the analysis. It is important that the values used for the facilities under study be reasonable. Also, if any characteristic is significantly different from the typical values used to develop Exhibit 16-14, particularly the weighted average *g/C* ratio and traffic signal spacing, the values taken from this exhibit will not be representative of the study facilities. In such cases, analysts are advised to develop their own generalized service volume tables by using representative local values or to proceed to a full operational analysis.

ACTIVE TRAFFIC MANAGEMENT STRATEGIES

Active traffic management (ATM) consists of the dynamic and continuous monitoring and control of traffic operations on a facility to improve facility performance. Examples of ATM measures on urban streets include congestion pricing zones, adaptive/responsive signal control, demand metering, changeable message signs, incident response, and work zone management. ATM measures can influence both the nature of the demand for the facility and the ability of the facility to deliver the capacity tailored to serve the demand. ATM measures can boost facility performance to the same extent as adding a conventional lane of capacity.

Other advanced design and management measures not included in the definition of ATM can also significantly boost facility performance. These non-ATM measures include lane treatments (bus lanes, bus streets, and reversible lanes), advanced interchange and intersection designs (divergent diamond interchanges, single-point urban interchanges, Michigan indirect left-turn intersections, and continuous flow intersections), and access management. These measures generally boost facility performance through the cost-effective addition of signal or lane capacity or both in unconventional ways to the facility.

More information on ATM measures and methods for their evaluation can be found in Chapter 35, Active Traffic Management.

4. EXAMPLE PROBLEMS

This part of the chapter describes the application of the automobile, pedestrian, bicycle, and transit methodologies through a series of example problems. Exhibit 16-15 provides an overview of these problems. The focus of the examples is to illustrate the multimodal facility evaluation process. An operational analysis level is used for all examples. 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 16-15
Example Problems

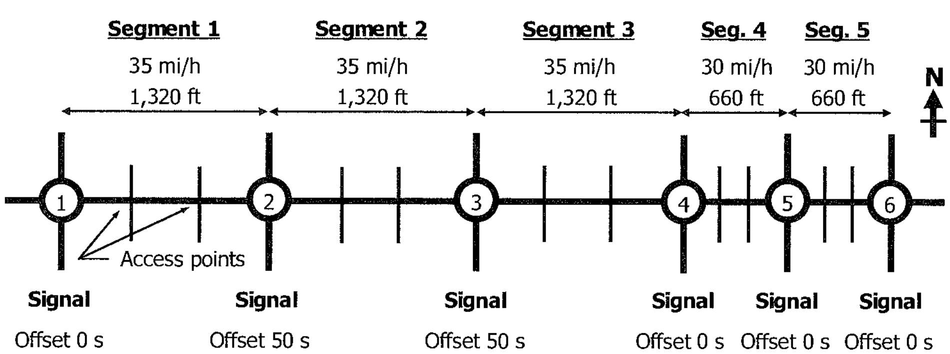
Problem Number	Description	Analysis Level
1	Auto-oriented urban street	Operational
2	Widen the sidewalks and add bicycle lanes on both sides of facility	Operational
3	Widen the sidewalks and add parking on both sides of facility	Operational

EXAMPLE PROBLEM 1: AUTO-ORIENTED URBAN STREET

The Urban Street Facility

A 1-mi urban street facility is shown in Exhibit 16-16. It is located in a downtown area and oriented in an east–west travel direction. The facility consists of five segments with a signalized boundary intersection for each segment. Segments 1, 2, and 3 are 1,320 ft long and have a speed limit of 35 mi/h. Segments 4 and 5 are 660 ft long and have a speed limit of 30 mi/h. Each segment has two access point intersections.

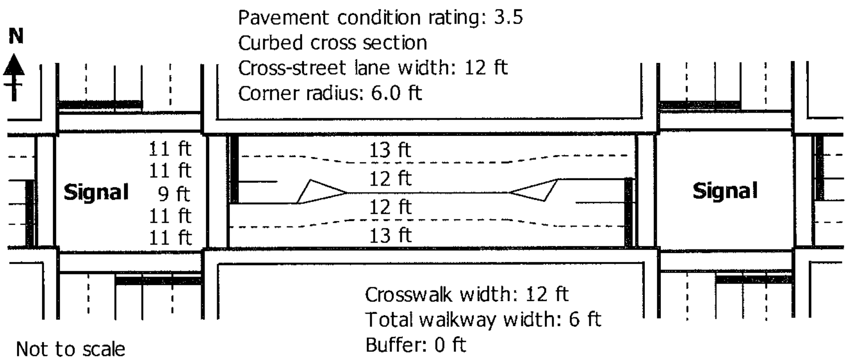
Exhibit 16-16
Example Problem 1: Urban Street Schematic



Segments 1, 2, and 3 pass through a mixture of office and strip commercial. Segments 4 and 5 are in a built-up shopping area.

The geometry of the typical street segment is shown in Exhibit 16-17. It is the same for each segment. The street has a curbed, four-lane cross section with two lanes in each direction. There is a 1.5-ft curb-and-gutter section on each side of the street. There are 200-ft left-turn bays on each approach to each signalized intersection. Right-turn vehicles share the outside lane with through vehicles on each intersection approach. A 6-ft sidewalk is provided on each side of the street adjacent to the curb. No fixed objects are located along the outside of the sidewalk. Midsegment pedestrian crossings are legal. No bicycle lanes are provided on the facility or its cross streets. No parking is allowed along the street.

Exhibit 16-17
Example Problem 1:
Segment Geometry



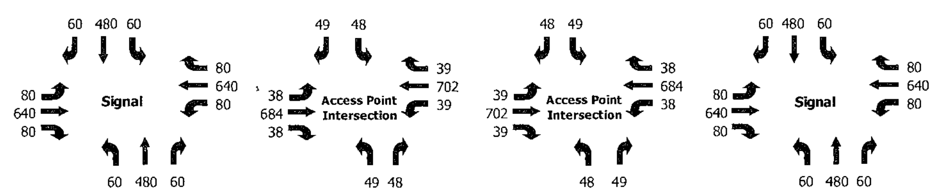
The Question

What are the travel speed and LOS of the automobile, pedestrian, bicycle, and transit modes in both directions of travel along the facility?

The Facts

The traffic counts for one segment are shown in Exhibit 16-18. The counts are the same for all of the other segments. The counts were taken during the 15-min analysis period of interest. However, they have been converted to hourly flow rates.

Exhibit 16-18
Example Problem 1:
Intersection Turn Movement
Counts



The signalization conditions are shown in Exhibit 16-19. The conditions shown are identified as belonging to Signalized Intersection 1; however, they are the same for the other signalized intersections (with exception of offset). The signals operate with coordinated–actuated control. The left-turn movements on the northbound and southbound approaches operate under permitted control. The left-turn movements on the major street operate as protected–permitted in a lead–lead sequence.

Exhibit 16-19 indicates that the passage time for each phase is 2.0 s. The minimum green setting is 5 s for the major-street left-turn phases and 18 s for the cross-street phases. The offset to Phase 2 (the reference phase) end-of-green interval is 0.0 s. The offset for each of the other intersections is shown in Exhibit 16-16. A fixed-force mode is used to ensure coordination is maintained. The cycle length is 100 s.

Geometric conditions and traffic characteristics for Signalized Intersection 1 are shown in Exhibit 16-20. They are the same for the other signalized intersections. The movement numbers follow the numbering convention shown in Exhibit 18-2 of Chapter 18.

Exhibit 16-19
Example Problem 1: Signal Conditions for Intersection 1

Signalized Intersection 1

General Information *19*

Cross Street:	First Avenue	**Analysis Period:**	AM peak

Phase Sequence and Left-Turn Mode

Major street sequence (movement numbers shown)	5 & 1 left leading	Cross street sequence (movement numbers shown)	No exclusive phase for 3 or 7
Major street left-turn mode (movement numbers shown)	5/1 Protected+Permitted	Cross street left-turn mode (movement numbers shown)	3/7 Permitted

Phase Settings

Approach	Eastbound		Westbound		Northbound		Southbound	
Phase number	5	2	1	6		8		4
Movement	L	T+R	L	T+R		L+T+R		L+T+R
Lead/lag left-turn phase	Lead	--	Lead	--		--		--
Left-turn mode	Pr/Pm	--	Pr/Pm	--		Perm.		Perm.
Passage time, s	2.0	--	2.0	--		2.0		2.0
Minimum green, s	5	--	5	--		18		18
Yellow + red clear, s	4.0	4.0	4.0	4.0		4.0		4.0
Phase split, s	20	45	20	45		35		35
Recall	No	--	No	--	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes

Ref. Phase	2	Offset, s:	0	Offset Ref.:	End of Green	Force Mode: Fixed
		Cycle, s:	100			

Enable Simultaneous Gap-Out?
Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑

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

Exhibit 16-20
Example Problem 1: Geometric Conditions and Traffic Characteristics for Signalized Intersection 1

Signalized Intersection 1

Signalized Intersection Input Data *(In each column, enter the volume and lanes data. For all other blue cells, enter values only if there is one or more lanes.)*

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
Volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
Lanes	1	2	0	1	2	0	1	2	0	1	2	0
Turn bay length, ft	200			200			200			200		
Sat. flow rate, veh/h/ln	1760	1829		1760	1829		1826	1838		1826	1838	
Platoon ratio	1.000	1.333		1.000	1.333		1.000	1.000		1.000	1.000	
Initial queue, veh	0	0		0	0		0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Stop line det. length, ft	40			40			40	40		40	40	
Max. allow. hdwy, s/ve	3.9			3.9			3.9	2.9		3.9	2.9	

The saturation flow rate is determined by using the procedure described in Chapter 18. All intersection movements include 3% heavy vehicles. The segment and intersection approaches are effectively level. No parking is allowed along the facility or its cross-street approaches. With a few exceptions (discussed below), local buses stop on the eastbound and westbound approaches to each signalized intersection at a rate of 3 buses/h.

Arrivals for all cross-street movements are effectively random, so a platoon ratio of 1.00 is used. The through movement arriving to the eastbound approach at Intersection 1 exhibits favorable progression from an upstream signal, so a platoon ratio of 1.33 is used. For similar reasons, a ratio of 1.33 is also used for the through movement arriving to the westbound approach at Intersection 6. Right-turn-on-red volume is estimated at 5.0% of the right-turn volume.

Each segment has a barrier curb along the outside of the street in each direction of travel. Allowing for the upstream signal width, the percentage of the segment length with curb is estimated at 94% for Segments 1, 2, and 3. It is estimated as 88% for Segments 4 and 5.

The traffic and lane assignment data for the two access point intersections for Segment 1 are shown in Exhibit 16-21. These data are the same for the other segments; however, the access point locations (shown in the first column) are reduced by one-half for Segments 4 and 5. The movement numbers follow the numbering convention shown in Exhibit 19-3 of Chapter 19, Two-Way STOP-Controlled Intersections. There are no turn bays on the segment at the two access point intersections.

Exhibit 16-21
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
440	Volume, veh/h	38	684	38	39	702	39	49	0	48	48	0	49
West end	Lanes	0	2	0	0	2	0	1	0	1	1	0	1
880	Volume, veh/h	39	702	39	38	684	38	48	0	49	49	0	48
	Lanes	0	2	0	0	2	0	1	0	1	1	0	1

A low wall is located along about 25% of the sidewalk in Segments 1, 2, and 3. In contrast, 10% of the sidewalk along Segments 4 and 5 is adjacent to a low wall, 35% to a building face, and 15% to a window display.

Office and strip commercial activity along Segments 1, 2, and 3 generates a pedestrian volume of 100 p/h on the adjacent sidewalks and crosswalks. Shopping activity along Segments 4 and 5 generates a pedestrian volume of 300 p/h on the adjacent sidewalks and crosswalks. A lack of bicycle lanes has discouraged bicycle traffic on the facility and its cross streets; however, a bicycle volume of 1.0 bicycle/h is entered for each intersection approach.

Local buses stop on the eastbound and westbound approaches to each signalized intersection, with the exception of Intersection 5. There are no stops on either approach to Intersection 5. However, transit stops are provided along the facility at 0.25-mi intervals, so the service is considered to be local. As a result, the westbound transit frequency on Segment 5 and the eastbound transit frequency on Segment 4 are considered to be the same as for the adjacent segments (i.e., 3 buses/h). The bus dwell time at each stop averages 20 s. Buses arrive within 5 min of their scheduled time about 75% of the time and have a load factor of 0.80 passengers/seat. Each bus stop has a bench but no shelter.

Outline of Solution

This section outlines the results of the facility evaluation. To complete this evaluation, the automobile, pedestrian, and bicycle methodologies in Chapter 18 were used to evaluate each of the signalized intersections on the facility. The procedure in Chapter 19 was used to estimate pedestrian delay when crossing at a midsegment location. The automobile, pedestrian, bicycle, and transit methodologies in Chapter 17 were then used to evaluate both directions of travel on each segment. Finally, the methodologies described in Section 2 were used to evaluate all four travel modes in both directions of travel on the facility. The findings from each evaluation are summarized in the following three subparts.

Intersection Evaluation

The results of the evaluation of Intersection 1 (i.e., First Avenue) are shown in Exhibit 16-22. The results for Intersections 2, 3, and eastbound Intersection 4 are similar. In contrast, Intersections 5 and 6 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Intersection 5 (i.e., Fifth Avenue) are shown in Exhibit 16-23. Intersection 6 and westbound Intersection 4 have similar results.

Exhibit 16-22
Example Problem 1: Intersection 1 Evaluation

Intersection	Intersection Evaluation Summary												
	Approach	Eastbound			Westbound			Northbound			Southbound		
First Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		100			100			100			100	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	2	0	1	2	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.17	0.34	0.34	0.15	0.34	0.34	0.36	0.62	0.62	0.36	0.62	0.62
20.4	Control delay, s/veh	7.78	5.75	5.77	7.04	13.38	13.73	43.18	34.24	34.28	43.18	34.24	34.28
Int. level of service	Stop rate, stops/veh	0.39	0.20	0.20	0.32	0.48	0.49	0.85	0.77	0.77	0.85	0.77	0.77
C	Level of service	**A**	**A**	**A**	**A**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	93.7			93.7			93.7			93.7		
	Crosswalk location	Crossing major			Crossing major			Crossing minor			Crossing minor		
	Crosswalk circulation area, ft2/p	75.9			75.9			82.4			82.4		
	Pedestrian delay, s/p	42.3			42.3			42.3			42.3		
	Pedestrian LOS score	2.75			2.75			2.66			2.66		
	Level of service	**C**			**C**			**B**			**B**		
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	n.a.			n.a.			n.a.			n.a.		
	Bicycle LOS score	3.72			3.72			2.87			2.87		
	Level of service	**D**			**D**			**C**			**C**		

Exhibit 16-23
Example Problem 1: Intersection 5 Evaluation

Intersection	Intersection Evaluation Summary												
	Approach	Eastbound			Westbound			Northbound			Southbound		
Fifth Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		300			300			300			300	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	2	0	1	2	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.16	0.34	0.34	0.16	0.34	0.34	0.36	0.62	0.62	0.36	0.62	0.62
20.2	Control delay, s/veh	7.73	8.74	8.31	7.63	9.84	9.48	43.10	34.13	34.17	43.10	34.13	34.17
Int. level of service	Stop rate, stops/veh	0.38	0.31	0.29	0.37	0.35	0.34	0.86	0.78	0.78	0.86	0.78	0.78
C	Level of service	**A**	**A**	**A**	**A**	**A**	**A**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	14.8			14.8			14.8			14.8		
	Crosswalk location	Crossing major			Crossing major			Crossing minor			Crossing minor		
	Crosswalk circulation area, ft2/p	24.5			24.5			26.7			26.7		
	Pedestrian delay, s/p	42.3			42.3			42.3			42.3		
	Pedestrian LOS score	2.70			2.70			2.62			2.62		
	Level of service	**B**			**B**			**B**			**B**		
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	n.a.			n.a.			n.a.			n.a.		
	Bicycle LOS score	3.72			3.72			2.87			2.87		
	Level of service	**D**			**D**			**C**			**C**		

Both exhibits indicate that the major-street vehicular through movements (i.e., eastbound Movement 2 and westbound Movement 6) operate with very low delay and few stops. The LOS is A and B for the eastbound and westbound through movements, respectively.

Pedestrian circulation area on the corners of Intersection 1 is generous, with pedestrians having the ability to move in their desired path without conflict. Corner circulation area at Intersection 5 is restricted, with pedestrians having very limited ability to pass slower pedestrians.

At Intersection 1, the low pedestrian volume results in generous crosswalk circulation area. Pedestrians rarely need to adjust their path to avoid conflicts. In contrast, the high pedestrian volume at Intersection 5 results in a constrained crosswalk circulation area. Pedestrians frequently adjust their path to avoid conflict. At each intersection, pedestrians experience an average wait of about 42 s at the corner to cross the street in any direction. This delay is lengthy, and some pedestrians may not comply with the signal indications. At Intersection 1, the pedestrian LOS is C for the major-street crossing and B for the minor-street crossing. At Intersection 5, the pedestrian LOS is B for the major-street and minor-street crossings.

Bicycle lanes are not provided at any intersection, so bicycle delay is not computed. The lack of a bicycle lane combined with a moderately high traffic volume results in a bicycle LOS D on the eastbound and westbound approaches of Intersection 1 and Intersection 5.

Segment Evaluation

The results of the evaluation of Segment 1 (i.e., First Avenue to Second Avenue) are shown in Exhibit 16-24. The results for Segments 2 and 3 are similar. In contrast, Segments 4 and 5 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Segment 5 (i.e., Fifth Avenue to Sixth Avenue) are shown in Exhibit 16-25. Segment 4 has similar results.

Exhibit 16-24
Example Problem 1:
Segment 1 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
First Avenue to Second Avenue	***Basic Description***		
	Speed limit, mi/h	35	35
	Vehicle volume, veh/h	800	800
	Through lanes, ln	2	2
Segment length, ft 1,320	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	40.3	40.3
	Travel speed, mi/h	23.8	23.2
	Spatial stop rate, stops/mi	1.77	1.92
	Level of service	**C**	**C**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	593.9	593.9
	Pedestrian travel speed, ft/s	3.54	3.54
	Pedestrian LOS score	3.76	3.76
	Level of service	**D**	**D**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.24	4.24
	Level of service	**D**	**D**
	Transit Level of Service		
	Transit travel speed, mi/h	12.7	12.5
	Transit LOS score	3.16	3.19
	Level of service	**C**	**C**

Exhibit 16-25
Example Problem 1:
Segment 5 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
Fifth Avenue to Sixth Avenue	***Basic Description***		
	Speed limit, mi/h	30	30
	Vehicle volume, veh/h	800	800
	Through lanes, ln	2	2
Segment length, ft 660	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	37.9	37.9
	Travel speed, mi/h	17.6	17.3
	Spatial stop rate, stops/mi	2.68	2.80
	Level of service	**D**	**D**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	153.3	153.3
	Pedestrian travel speed, ft/s	3.18	3.18
	Pedestrian LOS score	3.67	3.67
	Level of service	**D**	**D**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.48	4.48
	Level of service	**E**	**E**
	Transit Level of Service		
	Transit travel speed, mi/h	7.7	17.3
	Transit LOS score	3.64	2.79
	Level of service	**D**	**C**

Exhibit 16-24 indicates that the vehicular through movements on Segment 1 in the eastbound and westbound travel directions have a travel speed of 24 and 23 mi/h, respectively (i.e., about 58% of the base free-flow speed). The LOS of each movement is C. In contrast, Exhibit 16-25 indicates that the through movements have a travel speed of only about 17 mi/h on Segment 5 (or 46% of the base free-flow speed), which is LOS D. Vehicles stop at a rate of about 1.8 stops/mi on Segment 1 and about 2.7 stops/mi on Segment 5.

Pedestrian space on the sidewalk along the segment is generous on Segment 1 and adequate on Segment 5. These characterizations are based on

Exhibit 16-11 and an assumed dominance of platoon flow for Segments 4 and 5. Pedestrians on these sidewalks can walk freely without having to alter their path to accommodate other pedestrians. The segment travel speed (3.54 ft/s for Segment 1 and 3.18 ft/s for Segment 5) is adequate, but would desirably exceed 4.0 ft/s. Nevertheless, the sidewalk is near the traffic lanes and crossing the street at a midsegment location can be difficult. As a result, the pedestrian LOS is D on all segments.

Bicycle lanes are not provided along the segment, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a moderately high traffic volume results in a bicycle LOS D for both directions of travel on Segment 1. Bicycle service on Segment 5 is also poor. However, the short spacing between access points on Segment 5, relative to Segment 1, further degrades service quality such that the bicycle LOS on Segment 5 is E.

Transit travel speed is about 12 mi/h on Segment 1 and corresponds to LOS C. On Segment 5, the travel speed is about 8 mi/h and 17 mi/h in the eastbound and westbound directions, respectively. The low speed for the eastbound direction results in LOS D. The higher speed for the westbound direction is due to the lack of a westbound transit stop on Segment 5. It results in LOS C for this direction.

Facility Evaluation

The methodology described in Section 2 is used to compute the aggregate performance measures for each travel direction along the facility. The results are shown in Exhibit 16-26. This exhibit indicates that the vehicle travel speed is about 22 mi/h in each travel direction (or 56% of the base free-flow speed). An overall LOS C applies to both vehicular movements on the facility; however, it is noted that LOS D applies to Segments 4 and 5. Vehicles incur stops along the facility at a rate of about 1.9 stops/mi.

Exhibit 16-26
Example Problem 1: Facility Evaluation

	Facility Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
	Vehicle Level of Service		
	Base free-flow speed, mi/h	39.7	39.7
	Travel speed, mi/h	22.3	22.1
	Spatial stop rate, stops/mi	1.86	1.93
Facility length, ft	Level of service	**C**	**C**
5,280	Poorest perf. segment LOS	D	D
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	298.6	298.6
	Pedestrian travel speed, ft/s	3.4	3.4
	Pedestrian LOS score	3.73	3.74
	Level of service	**D**	**D**
	Poorest perf. segment LOS	D	D
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.30	4.30
	Level of service	**E**	**E**
	Poorest perf. segment LOS	E	E
	Transit Level of Service		
	Transit travel speed, mi/h	12.4	12.3
	Transit LOS score	3.15	3.16
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D

Pedestrian space on the sidewalk along the facility is generous. Pedestrians on the sidewalks can walk freely without having to alter their path to accommodate other pedestrians. The facility travel speed of about 3.4 ft/s is adequate, but would desirably exceed 4.0 ft/s. Nevertheless, the sidewalk is near the traffic lanes and crossing the street at a midsegment location can be difficult. As a result, the pedestrian LOS is D for both directions of travel.

Bicycle lanes are not provided along the facility, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a moderately high traffic volume results in an overall bicycle LOS E for both directions of travel.

Transit travel speed is about 12 mi/h on the facility in each direction of travel. An overall LOS C is assigned to each direction. The lower speed on westbound Segment 4 and eastbound Segment 5 is noted to result in LOS D for those segments.

EXAMPLE PROBLEM 2: PEDESTRIAN AND BICYCLE IMPROVEMENTS

The Urban Street Facility

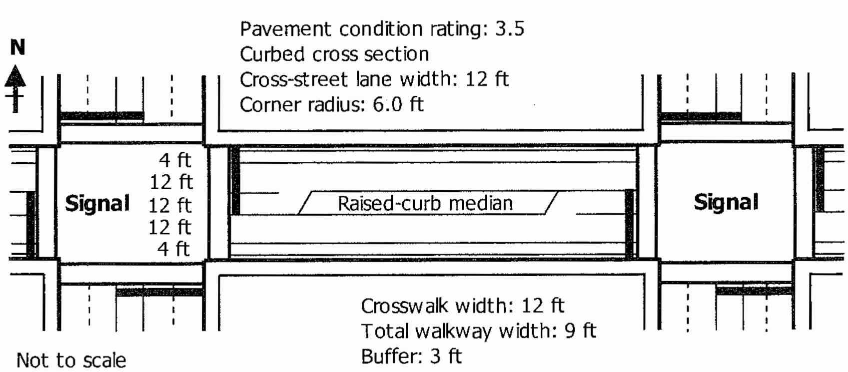
The 1-mi urban street facility shown in Exhibit 16-16 is being considered for geometric design modifications to improve pedestrian and bicycle service. The following changes to the facility are proposed:

- Eliminate one vehicle lane in each direction,
- Add a 12-ft raised-curb median,
- Add a 4-ft bicycle lane in each direction,
- Increase the total walkway width to 9 ft,
- Add a 3-ft buffer between the sidewalk and the curb, and
- Add bushes to the buffer using a 10-ft spacing.

No fixed objects are located along the outside of the sidewalk. The analysis for Example Problem 1 represents the existing condition, against which this alternative will be evaluated.

The geometry of the typical street segment is shown in Exhibit 16-27. It is the same for each segment. Additional segment details are provided in the discussion for Example Problem 1.

Exhibit 16-27
Example Problem 2:
Segment Geometry



The Question

What are the travel speed and LOS of the automobile, pedestrian, bicycle, and transit modes in both directions of travel along the facility?

The Facts

The traffic counts, signalization, and intersection geometry are listed in Exhibit 16-18 to Exhibit 16-21. They are unchanged from Example Problem 1.

Outline of Solution

This section outlines the results of the facility evaluation. To complete this evaluation, the automobile, pedestrian, and bicycle methodologies in Chapter 18 were used to evaluate each of the signalized intersections on the facility. The procedure in Chapter 19 was used to estimate pedestrian delay when crossing at a midsegment location. The automobile, pedestrian, bicycle, and transit methodologies in Chapter 17 were then used to evaluate both directions of travel on each segment. Finally, the methodologies described in Section 2 were used to evaluate all four travel modes in both directions of travel on the facility. The findings from each evaluation are summarized in the following three subparts.

Intersection Evaluation

The results of the evaluation of Intersection 1 (i.e., First Avenue) are shown in Exhibit 16-28. The results for Intersections 2, 3, and eastbound Intersection 4 are similar. In contrast, Intersections 5 and 6 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Intersection 5 (i.e., Fifth Avenue) are shown in Exhibit 16-29. Intersection 6 and westbound Intersection 4 have similar results.

Exhibit 16-28
Example Problem 2: Intersection 1 Evaluation

	Intersection Evaluation Summary												
Intersection	Approach		**Eastbound**			**Westbound**			**Northbound**			**Southbound**	
First Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		100			100			100			100	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	0	1	1	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.20	0.68	0.68	0.18	0.68	0.68	0.36	0.62	0.62	0.36	0.62	0.62
21.8	Control delay, s/veh	10.70	9.77	9.77	8.61	14.36	14.36	43.19	34.26	34.30	43.19	34.26	34.30
Int. level of service	Stop rate, stops/veh	0.54	0.24	0.24	0.45	0.46	0.46	0.85	0.77	0.77	0.85	0.77	0.77
C	Level of service	**B**	**A**	**A**	**A**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location		Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound	
	Corner circulation area, ft2/p		282.1			282.1			282.1			282.1	
	Crosswalk location			Crossing major			Crossing major			Crossing minor			Crossing minor
	Crosswalk circulation area, ft2/p			69.7			69.7			82.5			82.4
	Pedestrian delay, s/p			42.3			42.3			42.3			42.3
	Pedestrian LOS score			2.63			2.63			2.66			2.66
	Level of service			**B**			**B**			**B**			**B**
	Bicycle Level of Service												
	Bicycle delay, s/bicycle		8.3			8.3			n.a.			n.a.	
	Bicycle LOS score		2.99			2.99			2.77			2.77	
	Level of service		**C**			**C**			**C**			**C**	

Both exhibits indicate that the vehicular through movements on the facility (i.e., eastbound Movement 2 and westbound Movement 6) operate with low delay and few stops. For the eastbound through movement, the LOS is A at Intersection 1 and B at Intersection 5. The LOS is B for the westbound through movement at both intersections. Relative to Example Problem 1, the delay for the through movements has increased by a few seconds at Intersection 1 and by about 8 s at Intersection 5. This increase is sufficient to lower the LOS designation for the eastbound through movement at Intersection 5 (i.e., from A to B).

Exhibit 16-29
Example Problem 2:
Intersection 5 Evaluation

	Intersection Evaluation Summary												
Intersection	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Fifth Avenue	***Basic Description***												
	Applicable lane assignments	L	T	RT	L	T	RT	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		300			300			300			300	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	0	1	1	0	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.21	0.68	0.68	0.21	0.68	0.68	0.36	0.62	0.62	0.36	0.62	0.62
24.5	Control delay, s/veh	11.53	17.32	17.32	11.72	16.79	16.79	43.12	34.18	34.23	43.12	34.18	34.23
Int. level of service	Stop rate, stops/veh	0.59	0.58	0.58	0.59	0.56	0.56	0.86	0.78	0.78	0.86	0.78	0.78
C	Level of service	**B**	**B**	**B**	**B**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location	Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound		
	Corner circulation area, ft2/p	77.6			77.6			77.6			77.6		
	Crosswalk location			Crossing major			Crossing major			Crossing minor			Crossing minor
	Crosswalk circulation area, ft2/p			22.4			22.4			26.6			26.6
	Pedestrian delay, s/p			42.3			42.3			42.3			42.3
	Pedestrian LOS score			2.55			2.55			2.62			2.62
	Level of service			**B**			**B**			**B**			**B**
	Bicycle Level of Service												
	Bicycle delay, s/bicycle	8.3			8.3			n.a.			n.a.		
	Bicycle LOS score	2.99			2.99			2.77			2.77		
	Level of service	**C**			**C**			**C**			**C**		

Pedestrian circulation area on the corners of Intersections 1 and 5 is generous, with few instances of conflict. This condition is greatly improved from Example Problem 1 and reflects the provision of wider sidewalks.

Relative to Example Problem 1, the reduction in through lanes has reduced the time provided to pedestrians to cross the major street. This reduction resulted in larger pedestrian groups using the crosswalk and a small reduction in crosswalk pedestrian space. At Intersection 1, pedestrian space is still generous, with few instances of conflict. At Intersection 5, the problem is amplified by a higher pedestrian demand. Pedestrian space in the crosswalks is constrained, and pedestrians are likely to find that their ability to pass slower pedestrians is limited.

At each intersection, pedestrians experience an average wait of about 42 s at the corner to cross the street in any direction. This condition has not changed from Example Problem 1.

At both intersections, the pedestrian LOS is B for the major-street and minor-street crossings. Relative to Example Problem 1, the pedestrian LOS score has improved by about the same amount at all intersections. At Intersection 1, this change is sufficient to result in a change in service level (i.e., from C to B).

Bicyclists using the bicycle lanes are expected to be delayed about 8 s/bicycle on both the eastbound and westbound approaches to each intersection. This level of delay is desirably low. However, the bicycle lane is relatively narrow at 4 ft, which leads to LOS C on the eastbound and westbound approaches of both intersections. This LOS is noted to be an improvement over the LOS D identified in Example Problem 1.

Segment Evaluation

The results of the evaluation of Segment 1 (i.e., First Avenue to Second Avenue) are shown in Exhibit 16-30. The results for Segments 2 and 3 are similar. In contrast, Segments 4 and 5 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from the other intersections. The results for Segment 5 (i.e., Fifth Avenue to Sixth Avenue) are shown in Exhibit 16-31. Segment 4 has similar results.

Exhibit 16-30
Example Problem 2: Segment 1 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	Eastbound	Westbound
First Avenue to Second Avenue	***Basic Description***		
	Speed limit, mi/h	35	35
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 1,320	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	37.4	37.4
	Travel speed, mi/h	21.3	21.3
	Spatial stop rate, stops/mi	1.83	1.82
	Level of service	**C**	**C**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	809.9	809.9
	Pedestrian travel speed, ft/s	3.55	3.55
	Pedestrian LOS score	2.74	2.74
	Level of service	**B**	**B**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	13.18	13.18
	Bicycle LOS score	3.87	3.87
	Level of service	**D**	**D**
	Transit Level of Service		
	Transit travel speed, mi/h	10.3	10.4
	Transit LOS score	3.42	3.42
	Level of service	**C**	**C**

Exhibit 16-31
Example Problem 2: Segment 5 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	Eastbound	Westbound
Fifth Avenue to Sixth Avenue	***Basic Description***		
	Speed limit, mi/h	30	30
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 660	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	35.3	35.3
	Travel speed, mi/h	12.7	13.2
	Spatial stop rate, stops/mi	4.74	4.47
	Level of service	**E**	**E**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	225.4	225.4
	Pedestrian travel speed, ft/s	3.18	3.18
	Pedestrian LOS score	2.72	2.72
	Level of service	**B**	**B**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	11.75	11.75
	Bicycle LOS score	4.10	4.10
	Level of service	**D**	**D**
	Transit Level of Service		
	Transit travel speed, mi/h	5.2	13.2
	Transit LOS score	4.00	3.14
	Level of service	**D**	**C**

Exhibit 16-30 indicates that the vehicular through movements on Segment 1 in the eastbound and westbound travel directions have a travel speed of about 21 mi/h (i.e., about 57% of the base free-flow speed). LOS C applies to both movements. In contrast, Exhibit 16-31 indicates that the through movements have a travel speed of only about 13 mi/h on Segment 5 (or 37% of the base free-flow speed), which is LOS E. Vehicles stop at a rate of about 1.8 stops/mi on Segment 1 and about 4.6 stops/mi on Segment 5. Relative to Example Problem 1, conditions have notably degraded for vehicles traveling along Segment 5.

Pedestrian space on the sidewalk along the segment is generous on Segment 1. Pedestrians can walk freely without having to alter their path to accommodate other pedestrians. Pedestrian space is adequate on Segment 5, with pedestrians in platoons occasionally needing to adjust their path to avoid conflict. These characterizations are based on Exhibit 16-11 and an assumed dominance of platoon flow for Segments 4 and 5. Relative to Example Problem 1, the sidewalks are more distant from the traffic lanes and crossing the street at a midsegment location is easier because of the raised curb median. As a result, the pedestrian LOS is B on all segments.

Bicyclists using the bicycle lanes experience a travel speed of 13 mi/h on Segment 1 and 12 mi/h on Segment 5. This travel speed is considered desirable. However, the bicycle lane is relatively narrow at 4 ft, so a bicycle LOS D results for both directions of travel on each segment. While still poor, the bicycle LOS scores indicate that bicycle service has improved on both segments relative to that found in Example Problem 1. In fact, the bicycle LOS on Segment 5 has improved by one letter designation.

Transit travel speed is 10 mi/h on Segment 1 and corresponds to LOS C. On Segment 5, the travel speed is about 5 mi/h and 13 mi/h in the eastbound and westbound directions, respectively. The low speed for the eastbound direction results in LOS D. The higher speed for the westbound direction is due to the lack of a westbound transit stop on Segment 5. It results in LOS C. Relative to Example Problem 1, the slower vehicular travel speed has increased the transit LOS scores, but not enough to change the designated service level.

Facility Evaluation

The methodology described in Section 2 is used to compute the aggregate performance measures for each travel direction along the facility. The results are shown in Exhibit 16-32. This exhibit indicates that the vehicle travel speed is about 18 mi/h in each travel direction (or 49% of the base free-flow speed). An overall LOS D applies to vehicle travel in each direction on the facility. It is noted that LOS E applies to Segments 4 and 5. Vehicles incur stops along the facility at a rate of about 2.6 stops/mi. Relative to Example Problem 1, vehicular travel speed has dropped about 4 mi/h, and LOS has degraded one level for this scenario.

Exhibit 16-32
Example Problem 2: Facility Evaluation

	Facility Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
Facility length, ft 5,280	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	36.8	36.8
	Travel speed, mi/h	18.0	18.1
	Spatial stop rate, stops/mi	2.64	2.62
	Level of service	**D**	**D**
	Poorest perf. segment LOS	E	E
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	422.2	422.2
	Pedestrian travel speed, ft/s	3.4	3.4
	Pedestrian LOS score	2.74	2.74
	Level of service	**B**	**B**
	Poorest perf. segment LOS	B	B
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	12.8	12.8
	Bicycle LOS score	3.93	3.93
	Level of service	**D**	**D**
	Poorest perf. segment LOS	D	D
	Transit Level of Service		
	Transit travel speed, mi/h	9.3	9.3
	Transit LOS score	3.48	3.48
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D

Pedestrian space on the sidewalk along the facility is generous. Pedestrians on the sidewalks can walk freely without having to alter their path to accommodate other pedestrians. Increasing the separation between the sidewalk and traffic lanes and improving pedestrians' ability to cross the street at midsegment locations (by adding a raised-curb median) have resulted in an overall pedestrian LOS B for both directions of travel. This level compares favorably with the LOS D indicated in Example Problem 1.

Bicyclists in the bicycle lanes are estimated to experience an average travel speed of about 13 mi/h. This travel speed is considered desirable. However, the 4-ft bicycle lane is relatively narrow and produces LOS D. This level is one level improved over that found for Example Problem 1.

Transit travel speed is about 9 mi/h on the facility in each direction of travel. An overall LOS C is assigned to each direction. Relative to Example Problem 1, the LOS designation is unchanged; however, the transit speed is slower, and the transit LOS score indicates a small reduction in service.

EXAMPLE PROBLEM 3: PEDESTRIAN AND PARKING IMPROVEMENTS

The Urban Street Facility

The 1-mi urban street facility shown in Exhibit 16-16 is being considered for geometric design modifications to improve parking and pedestrian service. The following changes to the facility are proposed:

- Eliminate one vehicle lane in each direction,
- Add a 12-ft raised-curb median,
- Add a 9.5-ft parking lane in each direction, and
- Increase the total walkway width to 7 ft.

No fixed objects will be located along the outside of the sidewalk. The on-street parking is expected to be occupied 50% of the time. Parking maneuvers are estimated to cause 1.8 s/veh additional delay on Segments 1, 2, and 3. On Segments 4 and 5, these maneuvers are estimated to cause 0.3 s/veh additional delay. The analysis for Example Problem 1 represents the existing condition, against which this alternative will be evaluated.

The geometry of the typical street segment is shown in Exhibit 16-33. It is the same for each segment. Additional segment details are provided in the discussion for Example Problem 1.

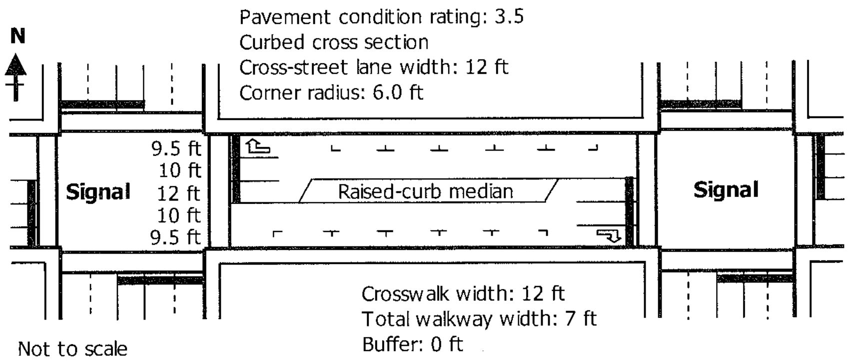
The Question

What are the travel speed and LOS of the automobile, pedestrian, bicycle, and transit modes in both directions of travel along the facility?

The Facts

The traffic counts, signalization, and intersection geometry are listed in Exhibit 16-18 to Exhibit 16-21. They are unchanged from Example Problem 1.

Exhibit 16-33
Example Problem 3:
Segment Geometry



Outline of Solution

This section outlines the results of the facility evaluation. To complete this evaluation, the automobile, pedestrian, and bicycle methodologies in Chapter 18 were used to evaluate each of the signalized intersections on the facility. The procedure in Chapter 19 was used to estimate pedestrian delay when crossing at a midsegment location. The automobile, pedestrian, bicycle, and transit methodologies in Chapter 17 were then used to evaluate both directions of travel on each segment. Finally, the methodologies described in Section 2 were used to evaluate all four travel modes in both directions of travel on the facility. The findings from each evaluation are summarized in the following three subparts.

Intersection Evaluation

The results of the evaluation of Intersection 1 (i.e., First Avenue) are shown in Exhibit 16-34. The results for Intersections 2, 3, and eastbound Intersection 4 are similar. In contrast, Intersections 5 and 6 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Intersection 5 (i.e., Fifth Avenue) are shown in Exhibit 16-35. Intersection 6 and westbound Intersection 4 have similar results.

Exhibit 16-34
Example Problem 3:
Intersection 1 Evaluation

	Intersection Evaluation Summary												
Intersection	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
First Avenue	***Basic Description***												
	Applicable lane assignments	L	T	R	L	T	R	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		100			100			100			100	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	1	1	1	1	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.19	0.59	0.09	0.16	0.59	0.09	0.36	0.62	0.62	0.36	0.62	0.62
21.8	Control delay, s/veh	10.01	7.99	9.00	7.40	16.60	11.49	43.19	34.26	34.30	43.19	34.26	34.30
Int. level of service	Stop rate, stops/veh	0.51	0.22	0.34	0.38	0.56	0.42	0.85	0.77	0.77	0.85	0.77	0.77
C	Level of service	**B**	**A**	**A**	**A**	**B**	**B**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location		Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound	
	Corner circulation area, ft2/p		148.1			148.1			148.1			148.1	
	Crosswalk location			Crossing major			Crossing major			Crossing minor			Crossing minor
	Crosswalk circulation area, ft2/p			74.0			74.0			82.6			82.4
	Pedestrian delay, s/p			42.3			42.3			42.3			42.3
	Pedestrian LOS score			2.67			2.67			2.66			2.66
	Level of service			**B**			**B**			**B**			**B**
	Bicycle Level of Service												
	Bicycle delay, s/bicycle		n.a.			n.a.			n.a.			n.a.	
	Bicycle LOS score		4.27			4.27			2.83			2.83	
	Level of service		**E**			**E**			**C**			**C**	

Both exhibits indicate that the vehicular through movements on the facility (i.e., eastbound Movement 2 and westbound Movement 6) operate with very low delay and few stops. For the eastbound through movement, the LOS is A at Intersection 1 and B at Intersection 5. The LOS is B for the westbound through movement at both intersections. Relative to Example Problem 1, the delay for the through movements has increased by a few seconds at both intersections. However, this increase is sufficient to lower the LOS designation for only the eastbound through movement at Intersection 5.

Exhibit 16-35
Example Problem 3: Intersection 5 Evaluation

	Intersection Evaluation Summary												
Intersection	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
Fifth Avenue	***Basic Description***												
	Applicable lane assignments	L	T	R	L	T	R	L	T	RT	L	T	RT
	Primary movement number	5	2	12	1	6	16	3	8	18	7	4	14
	Vehicle volume, veh/h	80	640	80	80	640	80	60	480	60	60	480	60
	Conflicting crosswalk volume, p/h		300			300			300			300	
	Bicycle volume, bicycle/h		1			1			1			1	
	Approach lanes, ln	1	1	1	1	1	1	1	2	0	1	2	0
	Vehicle Level of Service												
Int. delay, s/veh	Volume-to-capacity ratio	0.18	0.59	0.10	0.17	0.59	0.10	0.36	0.62	0.62	0.36	0.62	0.62
21.6	Control delay, s/veh	9.60	11.83	4.74	9.25	13.32	6.16	43.12	34.18	34.22	43.12	34.18	34.22
Int. level of service	Stop rate, stops/veh	0.51	0.39	0.19	0.49	0.45	0.24	0.86	0.78	0.78	0.86	0.78	0.78
C	Level of service	**A**	**B**	**A**	**A**	**B**	**A**	**D**	**C**	**C**	**D**	**C**	**C**
	Pedestrian Level of Service												
	Corner location		Adjacent to Eastbound			Adjacent to Westbound			Adjacent to Northbound			Adjacent to Southbound	
	Corner circulation area, ft2/p		33.0			33.0			33.0			33.0	
	Crosswalk location			Crossing major			Crossing major			Crossing minor			Crossing minor
	Crosswalk circulation area, ft2/p			23.8			23.8			26.6			26.6
	Pedestrian delay, s/p			42.3			42.3			42.3			42.3
	Pedestrian LOS score			2.61			2.61			2.62			2.62
	Level of service			**B**			**B**			**B**			**B**
	Bicycle Level of Service												
	Bicycle delay, s/bicycle		n.a.			n.a.			n.a.			n.a.	
	Bicycle LOS score		4.27			4.27			2.83			2.83	
	Level of service		**E**			**E**			**C**			**C**	

Pedestrian circulation area on the corners of Intersection 1 is generous. However, corner circulation area at Intersection 5 is constrained, with pedestrians frequently needing to adjust their path to avoid slower pedestrians. Regardless, this condition is improved from Example Problem 1 and reflects the provision of wider sidewalks.

Relative to Example Problem 1, the reduction in lanes has reduced the time provided to pedestrians to cross the major street. This reduction resulted in larger pedestrian groups using the crosswalk and a slight reduction in crosswalk pedestrian space. At Intersection 1, pedestrian space is generous. However, pedestrian space is constrained at Intersection 5, with pedestrians having limited ability to pass slower pedestrians as they cross the street.

At each intersection, pedestrians experience an average wait of about 42 s at the corner to cross the street in any direction. At both intersections, the pedestrian LOS is B for the major-street crossing and the minor-street crossing. The LOS designation has improved at Intersection 1 by one letter, relative to Example Problem 1, and remains unchanged at Intersection 5.

Bicycle lanes are not provided at any intersection, so bicycle delay is not computed. The lack of a bicycle lane combined with a high traffic volume results in a bicycle LOS E on the eastbound and westbound approaches of Intersection 1 and Intersection 5. This level is noted to be worse than the LOS D identified in Example Problem 1 because the traffic volume per lane has doubled.

Segment Evaluation

The results of the evaluation of Segment 1 (i.e., First Avenue to Second Avenue) are shown in Exhibit 16-36. The results for Segments 2 and 3 are similar. In contrast, Segments 4 and 5 are associated with a shorter segment length, lower speed limit, and higher pedestrian volume, so their operation is different from that of the other intersections. The results for Segment 5 (i.e., Fifth Avenue to Sixth Avenue) are shown in Exhibit 16-37. Segment 4 has similar results.

Exhibit 16-36
Example Problem 3: Segment 1 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	Eastbound	Westbound
First Avenue to Second Avenue	***Basic Description***		
	Speed limit, mi/h	35	35
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 1,320	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	37.4	37.4
	Travel speed, mi/h	20.0	19.5
	Spatial stop rate, stops/mi	2.05	2.22
	Level of service	**C**	**C**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	737.9	737.9
	Pedestrian travel speed, ft/s	3.55	3.55
	Pedestrian LOS score	2.89	2.89
	Level of service	**C**	**C**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.70	4.70
	Level of service	**E**	**E**
	Transit Level of Service		
	Transit travel speed, mi/h	10.5	10.2
	Transit LOS score	3.38	3.41
	Level of service	**C**	**C**

Exhibit 16-37
Example Problem 3: Segment 5 Evaluation

Segment	Segment Evaluation Summary		
	Travel Direction	Eastbound	Westbound
Fifth Avenue to Sixth Avenue	***Basic Description***		
	Speed limit, mi/h	30	30
	Vehicle volume, veh/h	800	800
	Through lanes, ln	1	1
Segment length, ft 660	***Vehicle Level of Service***		
	Base free-flow speed, mi/h	35.3	35.3
	Travel speed, mi/h	14.9	14.5
	Spatial stop rate, stops/mi	3.35	3.59
	Level of service	**D**	**D**
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	201.4	201.4
	Pedestrian travel speed, ft/s	3.18	3.18
	Pedestrian LOS score	2.87	2.87
	Level of service	**C**	**C**
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.94	4.94
	Level of service	**E**	**E**
	Transit Level of Service		
	Transit travel speed, mi/h	6.2	14.5
	Transit LOS score	3.84	3.01
	Level of service	**D**	**C**

Exhibit 16-36 indicates that the vehicular through movements on Segment 1 in the eastbound and westbound travel directions have a travel speed of about 20 mi/h (i.e., about 53% of the base free-flow speed). LOS C applies to both movements. In contrast, Exhibit 16-37 indicates that the through movements have a travel speed of only about 15 mi/h on Segment 5 (or 42% of the base free-flow speed), which is LOS D. Vehicles stop at a rate of about 2.1 stops/mi on Segment 1 and about 3.5 stops/mi on Segment 5. Relative to Example Problem 1, conditions have degraded for vehicles traveling along these segments, but not enough to drop the LOS designation.

Pedestrian space on the sidewalk along the segment is generous on Segment 1 and adequate on Segment 5. These characterizations are based on Exhibit 16-11 and an assumed dominance of platoon flow for Segments 4 and 5. Pedestrians on these sidewalks can walk freely without having to alter their path to accommodate other pedestrians. Relative to Example Problem 1, the sidewalks are more distant from the traffic lanes, and crossing the street at a midsegment location is easier because of the raised-curb median. As a result, the pedestrian LOS is improved on all segments (i.e., from LOS D to C).

Bicycle lanes are not provided along the segment, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a high traffic volume results in a bicycle LOS E for both directions of travel on Segment 1 and Segment 5. Relative to Example Problem 1, conditions have degraded for bicyclists on all segments, and the LOS for Segment 1 has dropped by one level. This reduction in service is due largely to the increased density of vehicles in the mixed traffic lanes.

Transit travel speed is about 10 mi/h on Segment 1 and corresponds to LOS C. On Segment 5, the travel speed is about 6 mi/h and 14 mi/h in the eastbound and westbound directions, respectively. The low speed for the eastbound direction results in LOS D. The higher speed for the westbound direction is due to the lack of a westbound transit stop on Segment 5. It results in LOS C. Relative to Example Problem 1 the slower travel speed has increased the transit LOS scores, but not enough to change the designated service level.

Facility Evaluation

The methodology described in Section 2 is used to compute the aggregate performance measures for each travel direction along the facility. The results are shown in Exhibit 16-38. This exhibit indicates that the vehicle travel speed is about 19 mi/h in each travel direction (or 50% of the base free-flow speed). An overall LOS C applies to both vehicular movements on the facility; however, it is noted that LOS D applies to Segments 4 and 5. Vehicles incur stops along the facility at a rate of about 2.3 stops/mi. Relative to Example Problem 1, vehicular LOS has degraded, but not enough to drop the LOS designation.

Exhibit 16-38
Example Problem 3: Facility Evaluation

	Facility Evaluation Summary		
	Travel Direction	**Eastbound**	**Westbound**
	Vehicle Level of Service		
	Base free-flow speed, mi/h	36.8	36.8
	Travel speed, mi/h	18.7	18.5
	Spatial stop rate, stops/mi	2.23	2.33
Facility length, ft	Level of service	**C**	**C**
5,280	Poorest perf. segment LOS	D	D
	Pedestrian Level of Service		
	Pedestrian space, ft2/p	381.1	381.1
	Pedestrian travel speed, ft/s	3.4	3.4
	Pedestrian LOS score	2.88	2.88
	Level of service	**C**	**C**
	Poorest perf. segment LOS	C	C
	Bicycle Level of Service		
	Bicycle travel speed, mi/h	No bicycle lane.	No bicycle lane.
	Bicycle LOS score	4.76	4.76
	Level of service	**E**	**E**
	Poorest perf. segment LOS	E	E
	Transit Level of Service		
	Transit travel speed, mi/h	10.2	10.1
	Transit LOS score	3.37	3.38
	Level of service	**C**	**C**
	Poorest perf. segment LOS	D	D

Pedestrian space on the sidewalk along the facility is generous. Pedestrians on the sidewalks can walk freely without having to alter their path to accommodate other pedestrians. Increasing the separation between the sidewalk and traffic lanes and improving pedestrians' ability to cross the street (by adding a raised-curb median) result in an overall pedestrian LOS C for both directions of travel. This level compares favorably with the LOS D indicated in Example Problem 1.

Bicycle lanes are not provided along the facility, so bicycle travel speed is not computed. The lack of a bicycle lane combined with a high traffic volume results in an overall bicycle LOS E for both directions of travel. Conditions have degraded slightly, relative to Example Problem 1, but not enough to drop the LOS designation.

Transit travel speed is about 10 mi/h on the facility in each direction of travel. An overall LOS C is assigned to each direction. Conditions have degraded slightly, relative to Example Problem 1, but not enough to drop the transit LOS designation.

5. REFERENCES

These references are available in the Technical Reference Library in Volume 4.

1. Bonneson, J. A., M. P. Pratt, and M. A. Vandehey. *Predicting the Performance of Automobile Traffic on Urban Streets: Final Report.* National Cooperative Highway Research Program Project 3-79. Texas Transportation Institute, Texas A&M University, College Station, Jan. 2008.
2. Dowling, R. G., D. B. Reinke, A. Flannery, P. Ryus, M. Vandehey, T. A. Petritsch, B. W. Landis, N. M. Rouphail, and J. A. Bonneson. *NCHRP Report 616: Multimodal Level of Service Analysis for Urban Streets.* Transportation Research Board of the National Academies, Washington, D.C., 2008.
3. Kittelson & Associates, Inc.; KFH Group, Inc.; Parsons Brinckerhoff Quade and Douglass, Inc.; and K. Hunter-Zaworski. *TCRP Report 100: Transit Capacity and Quality of Service Manual,* 2nd ed. Transportation Research Board of the National Academies, Washington, D.C., 2003.

CHAPTER 17
URBAN STREET SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

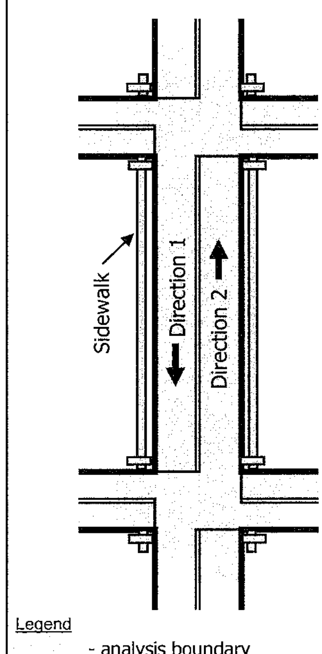
Chapter 17, Urban Street Segments, describes a methodology for evaluating the capacity and quality of service provided to road users traveling along an urban street segment. However, the methodology is much more than just a tool for evaluating capacity and quality of service. The methodology includes an array of performance measures that more fully describes segment operation for multiple travel modes. These measures serve as clues in identifying the source of problems and provide insight into the development of effective improvement strategies. The analyst is encouraged to consider the full range of measures when using this methodology.

OVERVIEW OF THE METHODOLOGY

This chapter's methodology is applicable to an urban or suburban street segment. The segment can be part of an arterial or collector street with one-way or two-way vehicular traffic flow. The intersections on the segment can be signalized or unsignalized.

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 backward from the intersection on each intersection leg. The size of this area is leg-specific and includes the most distant extent of any intersection-related queue expected to occur during the study period. For these reasons, the analysis boundaries should be established for each intersection on the basis of the conditions present during the analysis period. Practically speaking, the influence area should extend at least 250 ft back from the stop line on each intersection leg.



Analysis Level

Analysis level describes the level of detail used in applying the methodology. Three levels are recognized:

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

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

Study Period and Analysis Period

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

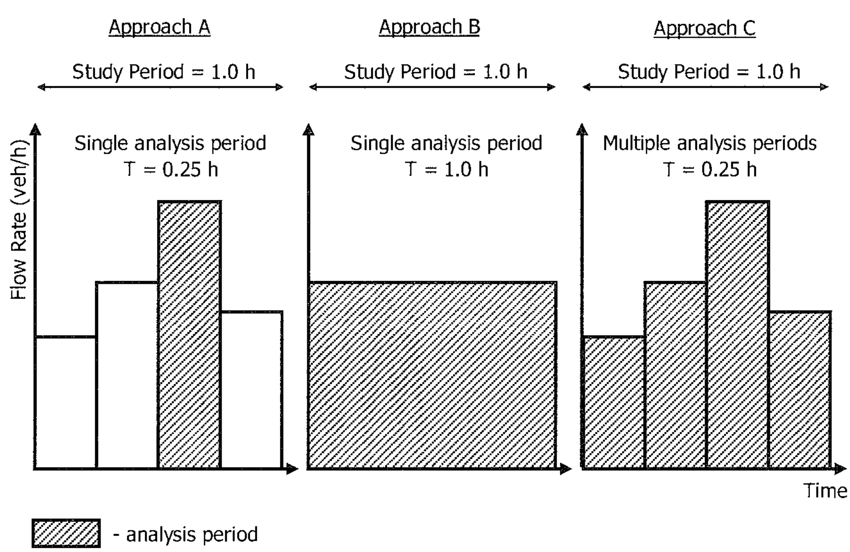
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the duration of the analysis period is in the range of 0.25 to 1 h. The longer durations in this range are sometimes used for planning 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.

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

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be 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.

Exhibit 17-1 demonstrates three alternative approaches an analyst might use for a given evaluation. Note that other alternatives exist and that the study period can exceed 1 h. Approach A is the one that has traditionally been used and, unless otherwise justified, is the one that is recommended for use.

Exhibit 17-1
Three Alternative Study Approaches



Approach A is based on the evaluation of the peak 15-min period during the study period. The analysis period T is 0.25 h. The equivalent hourly flow rate used for the analysis is based on either a peak 15-min traffic count multiplied by four or a 1-h demand volume divided by the peak hour factor. The former option is preferred whenever traffic counts are available. The peak hour factor equals the hourly count of vehicles divided by four times the peak 15-min count for a common hour interval. It is provided by the analyst or operating agency.

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. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay.

Performance Measures

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 automobile travel speed, automobile stop rate, automobile traveler perception score, pedestrian travel speed, pedestrian space, pedestrian perception score, bicycle travel speed, bicycle perception score, transit vehicle travel speed, transit wait–ride score, and transit passenger perception score.

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

Travel Modes

This chapter describes a separate methodology for evaluating urban street performance from the perspective of motorists, pedestrians, bicyclists, or transit passengers. These methodologies are referred to as the automobile methodology, pedestrian methodology, bicycle methodology, and transit methodology.

Each methodology consists of a set of procedures for computing the quality of service provided to one mode. Collectively, they can be used to evaluate the urban street segment operation from a multimodal perspective.

Each methodology 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 on the urban street are described in Chapters 18 to 22.

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

The transit methodology described in this chapter is applicable to the evaluation of passenger service provided by local public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. Nonlocal transit vehicle speed and delay are evaluated by using the automobile methodology.

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

Lane Groups and Movement Groups

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 Chapter 18, Signalized Intersections. They are also used in this chapter when the boundary intersection is signalized.

The automobile methodology in Chapter 18 is designed to evaluate the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group.* In general, a separate lane group is established for (*a*) each lane (or combination of adjacent lanes) that exclusively serves one movement and (*b*) each lane shared by two or more movements.

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

URBAN STREET SEGMENT DEFINED

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* represents the boundary between links and is represented by an intersection or ramp terminal. A *link* represents a length of roadway between two points. A link and its boundary points are referred to as a *segment.*

Previous editions of this manual have allowed the evaluation of one direction of travel along a segment (even when it served two-way traffic). This approach is retained in this chapter 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.

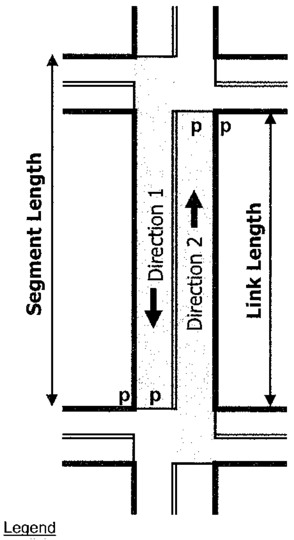
For the automobile methodology, a segment evaluation considers both directions of travel (when the street serves two-way traffic).

For the analysis of automobile 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 automobile methodology in this edition of the *Highway Capacity Manual* (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 these reasons, it is important to evaluate both travel directions on a two-way 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, then 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, then 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.

Segment Length Considerations

When a segment has a "short" length, then the interaction between traffic movements and traffic control devices at the two boundary intersections is sufficiently complex that a separate analysis of each element will 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, it is particularly complicated when the two intersections are signalized. The automobile methodology described in this chapter is not appropriate for the analysis of short segments. In contrast, the methodology described in Chapter 22, Interchange Ramp Terminals, is appropriate for the analysis of short segments at signalized interchanges.

It is difficult to define specific conditions under which a segment is short. However, two general rules apply in making this determination. First, a segment is considered to be short if the queue frequently extends back from one intersection into the other intersection (i.e., spills back) during the analysis period. Second, a segment is considered to be short if the through signal phase duration at the downstream intersection is longer than that needed to serve all the vehicles that store on the segment plus any vehicles that can enter it from the upstream signalized intersection while the downstream phase is green. This situation results in "demand starvation." It leads to the inefficient use of the downstream through phase and the retention of unserved vehicles on the approaches to the upstream intersection. In general, segments that are bounded by signalized intersections and are shorter than 400 ft may experience one or both of these conditions.

Platoons formed at a signalized intersection are typically dispersed by the time they reach a point about 0.6 mi downstream of the signal. This distance can vary depending on the amount of access point activity along the street and the speed of the traffic stream. Regardless, the influence of platoons on urban street operation is very likely to be negligible when segment length exceeds 2 mi. Therefore, if a segment exceeds 2 mi in length and its boundary points are signalized, then the analyst should evaluate the segment as an uninterrupted-flow highway segment with isolated intersections.

LOS CRITERIA

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

Automobile Mode

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

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.

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 85% 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 85% 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 17-2 lists the LOS thresholds established for the automobile mode on urban streets.

Exhibit 17-2
LOS Criteria: Automobile Mode

Travel Speed as a Percentage of Base Free-Flow Speed (%)	LOS by Volume-to-Capacity Ratio[a] ≤1.0	> 1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

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

Nonautomobile 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 when they assess 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 methodology for evaluating each mode provides a procedure for mathematically combining these factors into a score. This score is then used to determine the LOS that is provided for a given direction of travel along a segment.

Exhibit 17-3 lists the scores associated with each LOS for the pedestrian travel mode. The LOS for this particular mode 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 17-3
LOS Criteria: Pedestrian Mode

Pedestrian LOS Score	LOS by Average Pedestrian Space (ft^2/p)					
	>60	>40–60	>24–40	>15–24	>8.0–15[a]	≤ 8.0[a]
≤2.00	A	B	C	D	E	F
>2.00–2.75	B	B	C	D	E	F
>2.75–3.50	C	C	C	D	E	F
>3.50–4.25	D	D	D	D	E	F
>4.25–5.00	E	E	E	E	E	F
>5.00	F	F	F	F	F	F

Note: [a] In cross-flow situations, the LOS E/F threshold is 13 ft^2/p.

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.

Exhibit 17-4 lists the range of scores that are associated with each LOS for the bicycle and transit modes. This exhibit is also applicable for determining pedestrian LOS when a sidewalk is not available.

Exhibit 17-4
LOS Criteria: Bicycle and Transit Modes

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

REQUIRED INPUT DATA

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

Automobile Mode

This part describes the input data needed for the automobile methodology. The data are listed in Exhibit 17-5 and are identified as "input data elements." They must be separately specified for each direction of travel on the segment and for each boundary intersection.

The last column in Exhibit 17-5 indicates 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 18 to 22).

The data elements listed in Exhibit 17-5 do not include variables that are considered to represent calibration factors (e.g., acceleration rate). Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Exhibit 17-5
Input Data Requirements: Automobile Mode

Data Category	Location	Input Data Element	Basis
Traffic characteristics	Boundary intersection	Demand flow rate	Movement group
	Segment	Access point flow rate	Movement group
		Midsegment flow rate	Segment
Geometric design	Boundary intersection	Number of lanes	Movement group
		Upstream intersection width	Intersection
		Turn bay length	Segment approach
	Segment	Number of through lanes	Segment
		Number of lanes at access points	Segment approach
		Turn bay length at access points	Segment approach
		Segment length	Segment
		Restrictive median length	Segment
		Proportion of segment with curb	Segment
		Number of access point approaches	Segment
Other	Segment	Analysis period duration	Segment
		Speed limit	Segment
Performance measures	Boundary intersection	Through control delay	Through-movement group
		Through stopped vehicles	Through-movement group
		2nd- and 3rd-term back-of-queue size	Through-movement group
		Capacity	Movement group
	Segment	Midsegment delay	Segment
		Midsegment stops	Segment

Notes: Movement group = one value for each turn movement with exclusive lanes and one value for the through movement (inclusive of any turn movements in a shared lane).
Through-movement group = one value for the segment through movement at the downstream boundary intersection (inclusive of any turn movements in a shared lane).
Segment = one value or condition for each direction of travel on the segment.
Segment approach = one value or condition for each intersection approach on the subject segment.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 17-5. 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 18 to 22).

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. It is needed for all intersecting movements at each active access point intersection.

An access point approach is considered to be *active* if it has sufficient volume 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 vehicles per hour (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.

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

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, then 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, then it should be measured at several locations and an average used in the methodology.

There is one exception to the aforementioned definition of midsegment flow rate. Specifically, if a planning analysis is being conducted in which (*a*) the projected demand coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand can be divided by the 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.

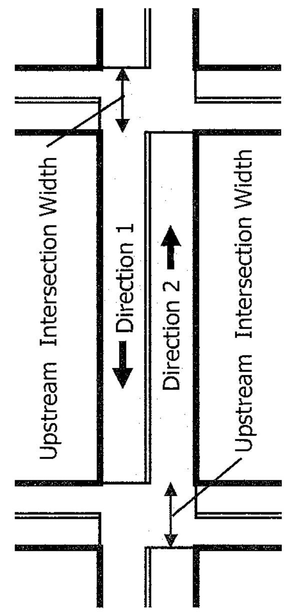
Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 17-5. These data describe the geometric elements of the segment or intersections that are addressed in the automobile methodology.

Number of Lanes

The number of lanes at the boundary intersection represents 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, then the turn movement is indicated to have 0 lanes.

Two-Way Vehicular Travel



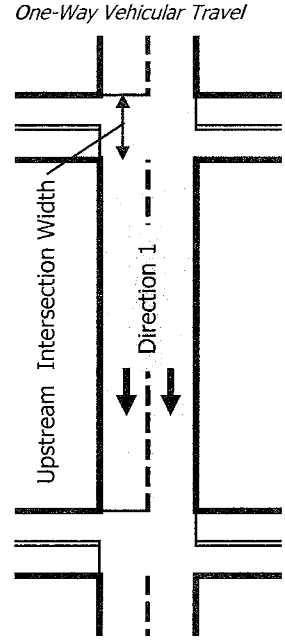
Upstream Intersection Width

The intersection width applies to the upstream boundary intersection for a given direction of travel and represents the effective width of the cross street. On a two-way street, it represents 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 represents the distance from the stop line to the far side of the most distant traffic lane on the cross street.

Turn Bay Length

Turn bay length represents 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, then the length entered should be an average value.

One-Way Vehicular Travel



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 their associated left-turning vehicles that store in the two-way left-turn lane.

Number of Through Lanes

The number of through lanes on the segment represents 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.

Number of Lanes at Access Points

The number of lanes at an access point intersection represents 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 number of lanes at boundary intersections.

Turn Bay Length at Access Points

Turn bay length represents 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.

Segment Length

Segment length represents 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, then 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 represents 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 represents that portion of the link length that has curb along the right side of the segment. This proportion is computed as the length of street with a curbed cross section divided by the link length. The length of street with a curbed cross section 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 value is input for each direction of travel along the segment.

Number of Access Point Approaches

The number of access point approaches along a segment represents the count of unsignalized driveway and public street approaches to the segment, regardless of the traffic demand entering the approach. This number is counted separately for each side of the segment. It must equal or exceed the number of active access points for which delay to segment through vehicles is computed. 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.

Other Data and Performance Measures

This subpart describes the data listed in Exhibit 17-5 that are categorized as "other data" or "performance measures."

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. Also, if the analysis period is other than 15 min, then the peak hour factor should not be used.

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

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

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

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

If a multiple-period analysis is used and the boundary intersections are signalized, then the procedure described in Chapter 18 should be used to guide the evaluation. When a procedure for multiple-period analysis is not provided in the chapter that corresponds to the boundary intersection configuration, the analyst should separately evaluate each period and use the residual queue from one period as the initial queue for the next period.

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

Speed Limit

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

The convenience of using speed limit as an input variable comes with a caution—the analyst must not infer a cause-and-effect relationship between the input speed limit and the estimated running speed. More specifically, the computed change in performance resulting from a change in the input speed limit is not likely to be indicative of performance changes that will actually be realized. Research indicates that a change in speed limit has a proportionally smaller effect on the actual average speed (*1*).

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 regarding specification of speed limits. If it is known that the posted speed limit does not satisfy these assumptions, then the speed limit value that is input to the methodology should be adjusted such that it is consistent with the assumptions.

Through Control Delay

The through control delay represents 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 18 to 22, 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, then 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. These variables are needed 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, then 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 represents 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 18 to 22, depending on the type of control used at the intersection. Chapter 19, 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 17-1.

Equation 17-1

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

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 19-43 in Chapter 19. 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 18 to 22 provides capacity by lane groups and the through movement is served in two or more lane groups, then 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.

Nonautomobile Modes

This part describes the input data needed for the pedestrian, bicycle, and transit methodologies. The data are listed in Exhibit 17-6 and are identified as "input data elements." They must be separately specified for each direction of travel on the segment.

Exhibit 17-6
Input Data Requirements: Nonautomobile Modes

Data Category	Location	Input Data Element	Pedestrian Mode	Bicycle Mode	Transit Mode
Traffic characteristics	Segment, transit	Dwell time			X
		Excess wait time			X
		Passenger trip length			X
		Transit frequency			X
		Passenger load factor			X
	Segment, other	Midsegment flow rate (motorized vehicles)	X	X	
		Percent heavy vehicles		X	
		Pedestrian flow rate	X		
		Prop. of on-street parking occupied	X	X	
Geometric design	Segment, roadway	Downstream intersection width	X		
		Segment length	X	X	X
		Number of through lanes	X	X	
		Width of outside through lane	X	X	
		Width of bicycle lane	X	X	
		Width of paved outside shoulder	X	X	
		Median type and curb presence	X	X	
		No. of access point approaches		X	
	Segment, sidewalk	Presence of a sidewalk	X		
		Total walkway width	X		
		Effective width of fixed objects	X		
		Buffer width	X		
		Spacing of objects in buffer	X		
Other	Segment	Area type			X
		Pavement condition rating		X	
		Distance to nearest signal-controlled crossing	X		
		Legality of midsegment pedestrian crossing	X		
		Proportion of sidewalk adjacent to window, building, or fence	X		
	Transit stop	Transit stop location			X
		Transit stop position			X
		Proportion of stops with shelters			X
		Proportion of stops with benches			X
Performance measures	Segment	Motorized vehicle running speed	X	X	X
		Pedestrian LOS score for link			X
	Boundary intersection	Through control delay			X
		Reentry delay			X
		Effective green-to-cycle-length ratio (if signalized)			X
		Volume-to-capacity ratio (if roundabout)			X
		Pedestrian delay	X		
		Bicycle delay		X	
		Pedestrian LOS score for intersection	X		
		Bicycle LOS score for intersection		X	

Exhibit 17-6 categorizes each input data element by travel mode methodology. An "X" is used to indicate the association between a data element and methodology. A blank cell indicates that the data element is not used as input for the corresponding methodology.

The data elements listed in Exhibit 17-6 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 the presentation of the methodology.

Traffic Characteristics Data

This subpart describes the traffic characteristics data listed in Exhibit 17-6. These data describe the vehicle, pedestrian, and transit traffic streams traveling along the segment during the analysis period. If there are multiple transit routes on the segment, then the transit-related variables are needed for each route.

Dwell Time

Dwell time represents 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* (3).

Excess Wait Time

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.

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, as 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, then Section 2, Methodology, provides a procedure for estimating excess wait time from on-time performance.

Passenger Trip Length

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. For most purposes, the average trip length can be determined from National Transit Database data for the transit agency (4) 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.

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 represents 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, then 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.

Midsegment Flow Rate

The midsegment flow rate of motorized vehicles is equivalent to the midsegment flow rate defined previously for the automobile mode.

Percent Heavy Vehicles

A heavy vehicle is defined as any vehicle with more than four tires touching the pavement. Local buses that stop within the intersection area are not included in the count of heavy vehicles. The percentage of heavy vehicles represents the count of heavy vehicles that arrive during the analysis period divided by the total vehicle count for the same period. This percentage is provided for the same location on the segment as represented by the midsegment flow rate.

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. Also, the use of pavement markings to delineate the parking lane should be noted.

If parking is not allowed on the segment, then the proportion equals 0.0. If parking is allowed along the segment but the spaces are not used during the analysis period, then 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, then the proportion equals 0.50.

Geometric Design Data

This subpart describes the geometric design data listed in Exhibit 17-6. These data describe the geometric elements that influence pedestrian, bicycle, or transit 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, number of through lanes, and number of access point approaches are defined previously for the automobile mode.

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 represents 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 represents 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, and Paved Outside Shoulder

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), and the outside shoulder. The outside shoulder may be used for on-street parking. The width of each of these elements is mutually exclusive because they are adjacent (i.e., not overlapped) in the cross section.

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).

Median Type and Curb Presence

The median type is designated as undivided, nonrestrictive (e.g., two-way left-turn lane), or restrictive (e.g., raised curb). Whether the cross section has curb on the outside edge of the roadway should also be noted.

Presence of a Sidewalk

A sidewalk is a paved walkway that is provided at the side of the roadway. It is assumed that pedestrians will 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, then 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 variable 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 23, 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 represents 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, then the average spacing of those objects that are 3 ft or more in height should also be recorded.

Other Data

This subpart describes the data listed in Exhibit 17-6 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."

Pavement Condition Rating

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 17-7 provides a description of pavement conditions associated with various ratings.

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 on 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.

Exhibit 17-7
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.

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 it is illegal to make this crossing at any point, then 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 proportion 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.

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 upon 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 upon exiting.

Proportion of Stops with Shelters and with Benches

These two input data describe the passenger amenities provided at a transit stop. A sheltered stop provides a structure with a roof and three enclosed 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 subpart describes the data listed in Exhibit 17-6 that are categorized as "performance measures." The through control delay variable was previously described for the automobile mode (in Exhibit 17-5).

Motorized Vehicle Running Speed

The motorized vehicle running speed is used in all of the nonautomobile methodologies. It is based on the segment running time obtained from the automobile methodology. The running speed is equal to the segment length divided by the segment running time.

Pedestrian LOS Score for Link

The pedestrian LOS score for the link is used in the transit methodology. It is obtained from the pedestrian methodology in this chapter.

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 (*3*):

- 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 19, 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 procedures of Chapter 18, 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.

Effective Green-to-Cycle-Length Ratio

The effective green-to-cycle-length ratio for the through movement is used in the transit methodology when the boundary intersection is a traffic signal and has a near-side transit stop. It is obtained from the Chapter 18 methodology.

Volume-to-Capacity Ratio (If Roundabout)

If the boundary intersection is a roundabout and it has a near-side transit stop, then the volume-to-capacity ratio for the rightmost lane of the segment approach to the roundabout is needed. It is obtained from the Chapter 21 methodology.

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 Chapter 18 is used to compute this delay.

The second delay variable needed 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, then the procedure described in Chapter 18 is used to compute this delay. If the nearest crossing is at a midsegment signalized crosswalk, then this delay should equal the pedestrian's average wait for service after pressing the pedestrian push button. This wait will depend on the signal settings and could range from 5 to 25 s/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, then the pedestrian waiting delay is determined by using the procedure in Chapter 19, Two-Way STOP-Controlled Intersections. If it is illegal, then the pedestrian waiting delay does not need to be calculated.

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 Chapter 18 is used to compute this delay.

Pedestrian LOS Score for Intersection

The pedestrian LOS score for the signalized intersection is used in the pedestrian methodology. It is obtained from the pedestrian methodology in Chapter 18.

Bicycle LOS Score for Intersection

The bicycle LOS score for the signalized intersection is used in the bicycle methodology. It is obtained from the bicycle methodology in Chapter 18.

SCOPE OF THE METHODOLOGY

Four methodologies are presented in this chapter. One methodology is provided for each of the automobile, pedestrian, bicycle, and transit modes. This section identifies the conditions for which each methodology is applicable.

- **Signalized and two-way STOP-controlled 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 automobile methodology can also be used to evaluate performance with all-way STOP- or YIELD-controlled (e.g., roundabout) boundary intersections.
- **Arterial and collector streets.** The four methodologies were developed with a focus on arterial and collector street conditions. If a methodology is used to evaluate a local street, then the performance estimates should be carefully reviewed for accuracy.
- **Steady flow conditions.** The four 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 four methodologies were developed to estimate the LOS perceived by automobile 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.
- **Target travel modes.** The automobile methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The pedestrian, bicycle, and transit methodologies address travel by walking, bicycle, and transit vehicle, respectively. The transit methodology is

limited to the evaluation of public transit vehicles operating in mixed or exclusive traffic lanes and stopping along the street. The methodologies are not designed to evaluate the performance of other travel means (e.g., grade-separated rail transit, golf carts, or motorized bicycles).

- **Influences in the right-of-way.** A road user's perception of quality of service is influenced by many factors inside and outside 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, or 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.
- **Mobility focus for automobile methodology.** The automobile methodology is intended to facilitate the evaluation of mobility. Accessibility to adjacent properties by way of automobile 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.
- **"Typical pedestrian" focus for pedestrian methodology.** The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. As such, the performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to 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.

LIMITATIONS OF THE METHODOLOGY

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

Automobile Modes

The automobile methodology does not directly account for the effect of the following conditions on street segment operation:

- On-street parking activity along the link (note that on-street parking activity on the approach to a signalized boundary intersection is addressed in Chapter 18, Signalized Intersections),
- Significant grade along the link,

- Capacity constraints between intersections (e.g., narrow bridges),
- Queuing at the downstream boundary intersection consistently backing up to and interfering with the operation of the upstream intersection or an access point intersection,
- 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.

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 18 to 22.

Nonautomobile Modes

This part identifies the limitations of the pedestrian, bicycle, and transit methodologies. These methodologies are not able to model the presence of railroad crossings. In addition, the pedestrian methodology does not model the following conditions:

- Segments bounded by all-way STOP-controlled intersections or roundabouts;
- 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; and
- Points where a high volume of vehicles cross the sidewalk, such as a parking garage entrance.

In addition, the bicycle methodology is not able to model the following conditions:

- Segments bounded by all-way STOP-controlled intersections or roundabouts, and
- Grades in excess of 2%.

With regard to the first bullet point in each of the two lists above, procedures have not been developed yet to address the effect of all-way STOP control or YIELD control on intersection performance from a pedestrian or bicyclist perspective.

2. METHODOLOGY

OVERVIEW

This section describes four methodologies for evaluating the performance of an urban street segment. Each methodology addresses one possible travel mode within the street right-of-way. Analysts should choose the combination of methodologies that are appropriate for their analysis needs.

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

The focus of each methodology in this chapter is the segment. Methodologies for quantifying the performance of the downstream boundary intersection are described in other chapters (i.e., Chapters 18 to 22). 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 each mode and travel direction.

AUTOMOBILE MODE

This subsection provides an overview of the methodology for evaluating urban street segment performance from the motorist's perspective. 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. Default values are provided in Section 3, Applications, to support planning analyses for which the required input data are not available.

A Quick Estimation Method for evaluating segment performance at a planning level of analysis is provided in Chapter 30, Urban Street Segments: Supplemental. This method is not computationally intense and can be applied by using hand calculations.

The methodology is used to evaluate automobile performance on an urban street segment. 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 automobile 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 automobile performance in noncoordinated systems.

Because of the intensity of the computations for coordinated–actuated control, the objective of this subsection 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 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 Chapter 30.

Framework

Exhibit 17-8 illustrates the calculation framework of the automobile methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. These calculations are described more fully in the remainder of this subsection.

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.

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.

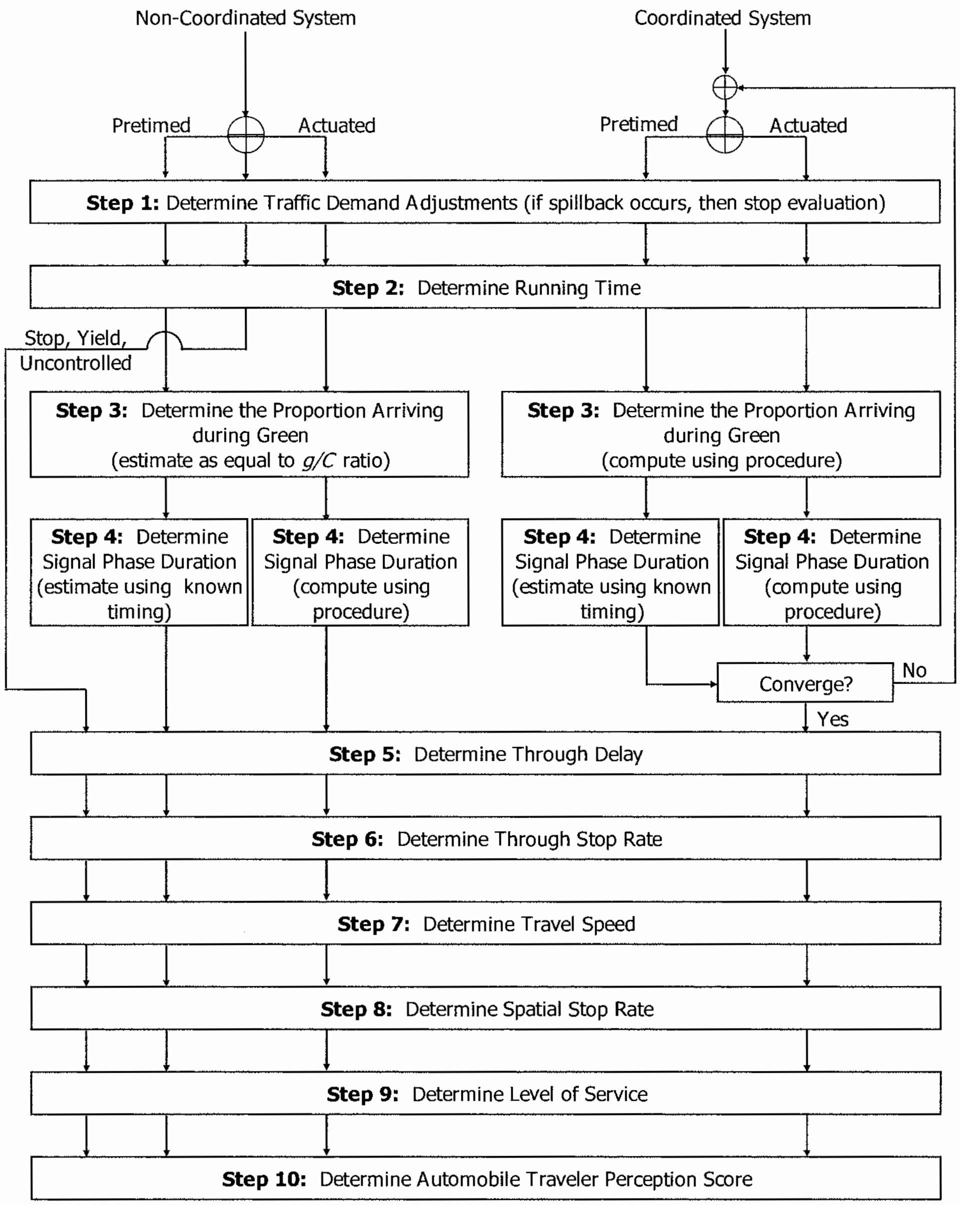
There is reference in Exhibit 17-8 to various procedures described in Chapters 18, 20, and 21. With regard to Chapter 18, the procedure for computing actuated phase duration is needed for the analysis of actuated intersections on both coordinated and noncoordinated segments. Also, the procedure for computing control delay in Chapter 18 is needed for the estimation of segment through-movement delay. The delay estimation procedure for roundabouts and all-way STOP-controlled intersections is needed from their respective chapters for the analysis of noncoordinated segments.

Performance measures estimated for each segment travel direction include

- Travel speed,
- Stop rate, and
- Automobile traveler perception score.

The perception score is derived from traveler perception research and is an indication of travelers' relative satisfaction with service provided along the segment.

Exhibit 17-8
Automobile Methodology for Urban Street Segments



Step 1: Determine Traffic Demand Adjustments

During this step, various adjustments are undertaken to ensure the volumes evaluated accurately reflect segment traffic conditions. The adjustments include (*a*) limiting entry to the segment due to 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. As indicated in Exhibit 17-8, the evaluation should not proceed if spillback occurs because the methodology does not address this condition.

The procedures for making these adjustments and checks are described in Chapter 30. These adjustments and checks are not typically used for planning and preliminary engineering analyses.

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, then 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 is taken at a different time than the count at the adjacent intersection. They are also likely to exist 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 such 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 17-9. There are three entry volumes at upstream Intersection A that contribute to three exit volumes at downstream Intersection B. There is also an entrance and exit volume at the access point intersection located between the two intersections. It should be noted that 1,350 veh/h enter the segment and 1,350 veh/h exit 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 17-10.

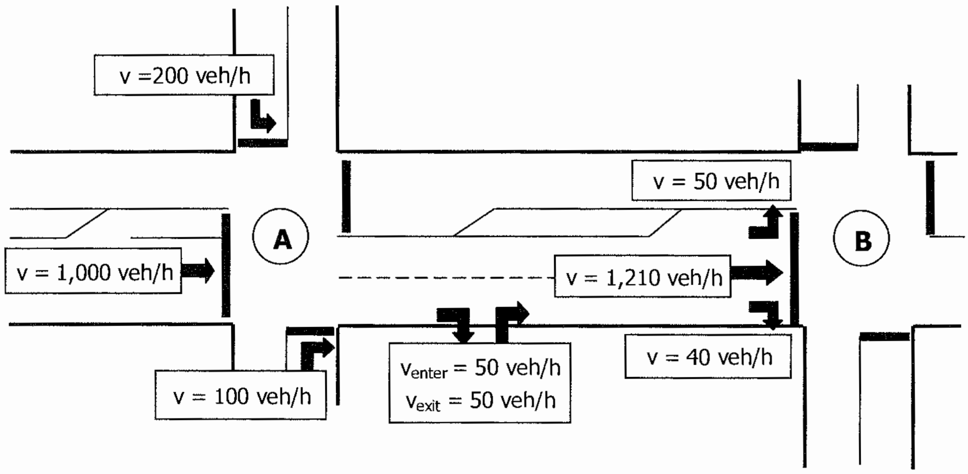


Exhibit 17-9
Entry and Exit Volume on Example Segment

Exhibit 17-10
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 17-10 correspond to the entry volumes shown in Exhibit 17-9. The row totals in the last column of Exhibit 17-10 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 automobile 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.

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 segment and bay spillback must be checked during this step.

Use of this methodology to evaluate segments (or intersection turn bays) with significant, sustained spillback is problematic because of the associated unsteady conditions and complex interactions. The procedure described in Chapter 30 is used in this step to compute the time when sustained spillback occurs, if it occurs. If this time of occurrence is shorter than the analysis period, then the methodology may not yield accurate performance estimates. In this situation, the analyst should consider either (*a*) reducing the analysis period such that it ends before spillback occurs or (*b*) using an alternative analysis tool that is able to 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 represents the average running speed of through automobiles 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, and curb presence. It is computed by using Equation 17-2. Alternatively, it can be measured in the field by using the technique described in Chapter 30.

$$S_{fo} = S_0 + f_{CS} + f_A$$

Equation 17-2

where

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

S_0 = speed constant (mi/h),

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

f_A = adjustment for access points (mi/h).

The speed constant and adjustment factors used in Equation 17-2 are listed in Exhibit 17-11. Equations provided in the table footnote can also be used to compute these adjustment factors.

Exhibit 17-11
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] 1 Lane	2 Lanes	3 Lanes	4 Lanes
0	0.0	0.0	0.0	0.0
2	-0.2	-0.1	-0.1	0.0
4	-0.3	-0.2	-0.1	-0.1
10	-0.8	-0.4	-0.3	-0.2
20	-1.6	-0.8	-0.5	-0.4
40	-3.1	-1.6	-1.0	-0.8
60	-4.7	-2.3	-1.6	-1.2

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); and W_i = width of signalized intersection (ft).

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 17-3 is used to compute the value of an adjustment factor that accounts for this influence.

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

Equation 17-3

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 17-3 was derived by using signalized boundary intersections. For more general applications, the definition of distance L_s is broadened such 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.

Free-Flow Speed

Free-flow speed is computed by using Equation 17-4 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.

Equation 17-4

$$S_f = S_{fo} f_L$$

where S_f equals the free-flow speed (mi/h) and other variables are as previously defined.

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 17-5 is used to compute the proximity adjustment factor.

Equation 17-5

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

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 17-12. 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, it is unlikely that a volume in excess of this amount will be experienced on a segment bounded by intersections at which the through movement is regulated by a traffic control device.

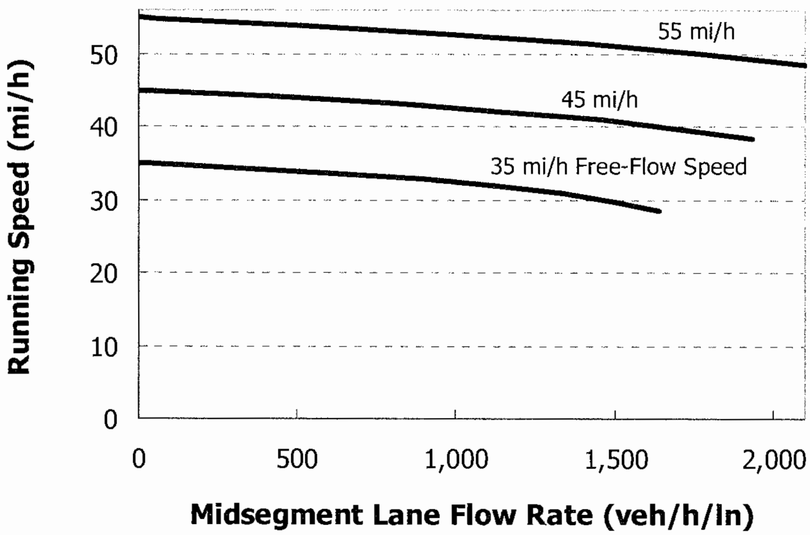


Exhibit 17-12
Speed–Flow Relationship for Urban Street Segments

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 also 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 Chapter 30.

For planning and preliminary engineering analyses, Exhibit 17-13 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 17-6).

Exhibit 17-13
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 17-13 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%, then the delays can be reduced proportionally. For example, if the subject access point has 5% left turns and 5% right turns, then 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, then the values listed in the exhibit should be multiplied by 0.5. If both turn movements are provided a bay of adequate length, then the delay due to turns can be assumed to equal 0.0 seconds per vehicle per access point (s/veh/pt).

D. Estimate Delay due to Other Sources

Numerous other factors could cause a driver to reduce speed or to incur delay while traveling along a segment. 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.

Of the many sources for midsegment delay, the automobile 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, then it can be included in the equation to compute running time.

E. Compute Segment Running Time

Equation 17-6 is used to compute segment running time based on consideration of through movement control at the boundary intersection, free-flow speed, vehicle proximity, and various midsegment delay sources.

Equation 17-6

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

with

Equation 17-7

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

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).

Other variables are as previously defined. The variables l_1, f_x, v_{th}, and c_{th} used with the first term in Equation 17-6 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, then 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). This procedure is described in this step. The procedure described in Chapter 18, Signalized Intersections, should be used 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), then 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, then 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, then 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 17-8. 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, major-street through, and 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 17-14 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 Chapter 30.

The gray shaded area in Exhibit 17-14 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 by using Equation 17-8.

Equation 17-8

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

where

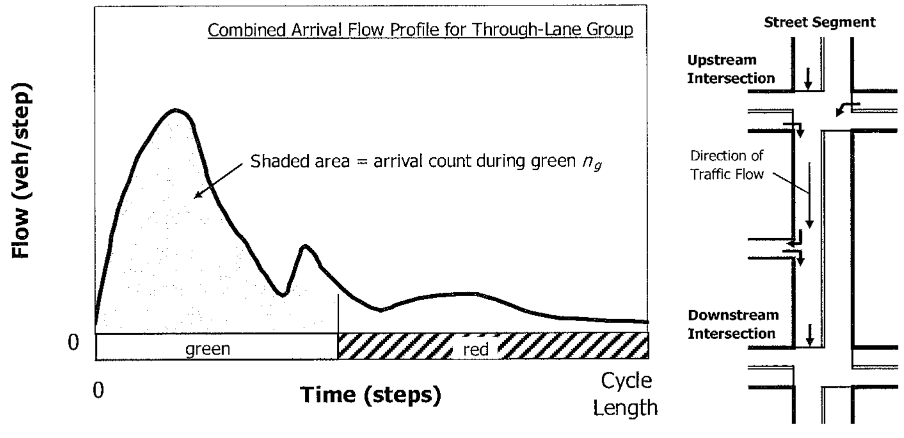
P = proportion of vehicles arriving during the green indication,

n_g = arrival count during green (veh),

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

C = cycle length (s).

Exhibit 17-14
Use of an Arrival Flow Profile to Estimate the Volume Arriving During Green



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, then this step is skipped.

If the downstream boundary intersection has pretimed signal control, then the signal phase duration is an input value. If this intersection has some form of actuated control, then the procedure described in Chapter 18 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 represents the sum of two delay sources. One source is the delay due to the traffic control at the boundary intersection. It is called control delay. The other delay is that due to the negotiation of intersection geometry, such as curvature. It is called geometric delay.

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

- Signal control (Chapter 18 or 22),
- All-way STOP control (Chapter 20), and
- YIELD control at a roundabout intersection (Chapter 21).

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, then 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 circular intersection is not negligible and should be added to the control delay to obtain the necessary through delay. A procedure for estimating geometric delay for roundabout intersections is described in Chapter 33, Roundabouts: Supplemental.

If the segment is not in a coordinated system, the through delay estimate should be based on isolated operation. The methodologies in Chapters 18 to 21 can be used to provide this estimate.

If the segment is within a coordinated signal system, then the methodology in Chapter 18 or Chapter 22 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 procedure for calculating this factor is described in Section 1 of Chapter 18.

If the through movement shares one or more lanes at a signalized boundary intersection, then the through delay is computed by using Equation 17-9.

Equation 17-9

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

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 Chapter 18, Signalized Intersections, is used to estimate the variables shown in Equation 17-9.

Step 6: Determine Through Stop Rate

As with control delay, 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 to through vehicles from those incurred by nonthrough vehicles. However, these 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, then 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 17-10.

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

Equation 17-10

with

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

Equation 17-11

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

Equation 17-12

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

Equation 17-13

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).

Other variables are as previously defined. The procedure for computing N_f, Q_2, and Q_3 is provided in Chapter 31, Signalized Intersections: Supplemental.

The first term in Equation 17-10 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, then $N_{f,i}$ 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 17-10 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 17-14 is used to compute the travel speed for the subject direction of travel along the segment.

Equation 17-14

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

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 control delay used in Equation 17-14 is that incurred by the through-lane group at the downstream boundary intersection.

Step 8: Determine Spatial Stop Rate

Equation 17-15 is used to compute the spatial stop rate for the subject direction of travel along the segment.

Equation 17-15

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

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 17-15 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, then it can be included in the calculation by using the variable h_{other}.

Step 9: Determine LOS

LOS is determined for both directions of travel along the segment. Exhibit 17-2 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, expressed as a percentage of the base free-flow speed. The second measure is the volume-to-capacity ratio for the through movement at the downstream boundary intersection.

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 attributed to each direction of travel applies to the segment, which includes both the link and the downstream boundary intersection. Chapters 18 to 22 describe LOS thresholds for the boundary intersection. The automobile methodology does not assign a LOS indicator to 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 by using Equation 17-16 to Equation 17-21.

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

Equation 17-16

with

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

Equation 17-17

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

Equation 17-18

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

Equation 17-19

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

Equation 17-20

Equation 17-21

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

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).

Other variables are as previously defined. The derivation of Equation 17-16 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 17-16 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 17-16 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.

PEDESTRIAN MODE

This subsection describes the methodology for evaluating the performance of an urban street segment in terms of its service to pedestrians.

Urban street segment 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, then it is assumed that pedestrians will walk in the street on that side (even if there is a sidewalk on the other side).

The methodology is focused on the analysis of a segment with either signal-controlled or two-way STOP-controlled boundary intersections. Chapter 18 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.

The pedestrian methodology is applied through a series of nine steps that culminate in the determination of the segment LOS. These steps are illustrated in Exhibit 17-15. Performance measures that are estimated include

- Pedestrian travel speed,
- Average pedestrian space, and
- Pedestrian LOS scores for the link and segment.

A methodology for evaluating off-street pedestrian facilities is provided in Chapter 23, Off-Street Pedestrian and Bicycle Facilities.

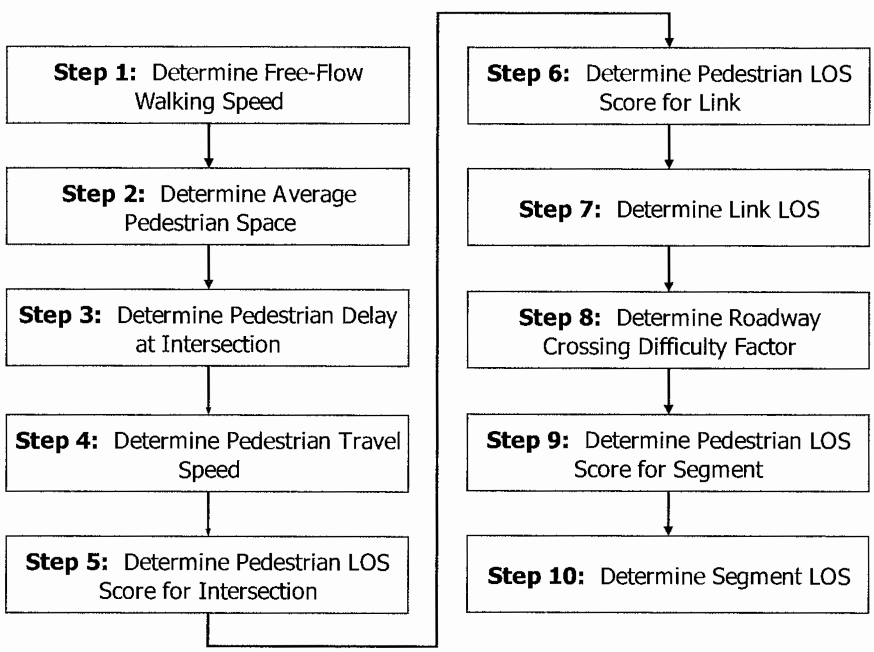


Exhibit 17-15
Pedestrian Methodology for Urban Street Segments

Link-Based Evaluation

Steps 6 and 7 of the pedestrian methodology can be used as a stand-alone procedure 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., crossing difficulty or 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 fully reflect segment performance.

Concepts

The methodology provides a variety of measures for evaluating segment performance in terms of its service to pedestrians. Each measure describes a different aspect of the pedestrian trip along the segment. One measure is the LOS score. This score is an indication of the typical pedestrian's perception of the overall segment travel experience. A second measure is the average speed of pedestrians traveling along the segment.

A third measure is based on the concept of "circulation area." It represents 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 17-16 provides a qualitative description of pedestrian space that can be used to evaluate sidewalk performance from a circulation-area perspective.

Exhibit 17-16
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 17-16 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.

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 (*6*). 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 17-22 to Equation 17-26.

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

Equation 17-22

with

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

Equation 17-23

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

Equation 17-24

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

Equation 17-25

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

Equation 17-26

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),

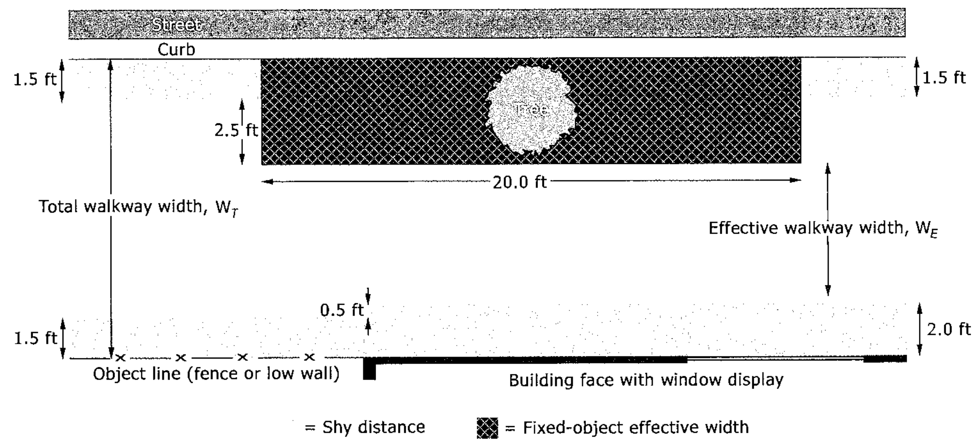
p_{fence} = proportion of sidewalk length adjacent to a fence or low wall (decimal),

$w_{O,i}$ = 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 17-17.

Exhibit 17-17
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 23, 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 17-17. 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 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 17-27 for the subject sidewalk. The variable v_{ped} is an input variable.

Equation 17-27

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

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 17-28. This equation is derived from the relationship between flow rate and average walking speed described in Exhibit 23-1 of Chapter 23.

Equation 17-28

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

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

D. Compute Pedestrian Space

Finally, Equation 17-29 is used to compute average pedestrian space.

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

Equation 17-29

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

The pedestrian space obtained from Equation 17-29 can be compared with the ranges provided in Exhibit 17-16 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 represents an input variable for the methodology and is described in Section 1, Required Input Data.

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

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 plus the time required to walk the length of the segment. As such, it is typically slower than the average walking speed. The pedestrian travel speed is computed by using Equation 17-30.

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

Equation 17-30

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 when 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, then the pedestrian methodology described in Chapter 18 is used for this determination. If the boundary intersection is two-way STOP controlled, then the score is equal to 0.0.

Step 6: Determine Pedestrian LOS Score for Link

The pedestrian LOS score for the link $I_{p,link}$ is calculated by using Equation 17-31.

Equation 17-31

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

with

Equation 17-32

$$F_w = -1.2276 \ln(W_v + 0.5\, W_1 + 50\, p_{pk} + W_{buf}\, f_b + W_{aA}\, f_{sw})$$

Equation 17-33

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

Equation 17-34

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

where

$I_{p,link}$ = 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 17-18) (ft);

W_1 = effective width of combined bicycle lane and shoulder (see Exhibit 17-18) (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_{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 17-32 to Equation 17-34 is dependent on various conditions. These conditions are identified in Column 1 of Exhibit 17-18. If the condition is satisfied, then the equation in Column 2 is used to compute the variable value. If it is not satisfied, then 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 17-18
Variables for Pedestrian LOS Score for Link

Condition	Variable When Condition Is Satisfied	Variable When Condition Is Not Satisfied
$p_{pk} = 0.0$	$W_t = W_{ol} + W_{bl} + W_{os}^*$	$W_t = W_{ol} + W_{bl}$
$v_m > 160$ veh/h or street is divided	$W_v = W_t$	$W_v = W_t(2 - 0.005\ v_m)$
$p_{pk} < 0.25$ or parking is striped	$W_1 = W_{bl} + W_{os}^*$	$W_1 = 10$

Notes: W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);
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); and
W_{bl} = width of the bicycle lane = 0.0 if bicycle lane not provided (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 17-17 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; it is also sensitive 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, then 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 17-31. 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 $I_{p,link}$ 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 automobile methodology, as described in a previous subsection.

Step 7: Determine Link LOS

The pedestrian LOS for the link is determined by using the pedestrian LOS score from Step 6 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit

17-3 to determine the LOS for the specified direction of travel along the subject link. If a sidewalk does not exist and pedestrians are relegated to walking in the street, then LOS is determined by using Exhibit 17-4 because the pedestrian space concept does not apply.

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 it may be 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, then the pedestrian crosses when there is an acceptable gap in the motorized vehicle stream.

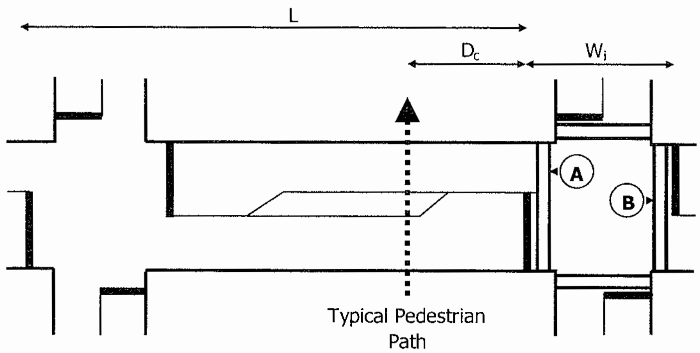
Each of these two crossing options is considered in this step, with that option requiring the least delay used as the basis for computing the pedestrian roadway crossing difficulty factor. The time to walk across the segment is common to both options and therefore is not included in the delay estimate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

A. Compute Diversion Delay

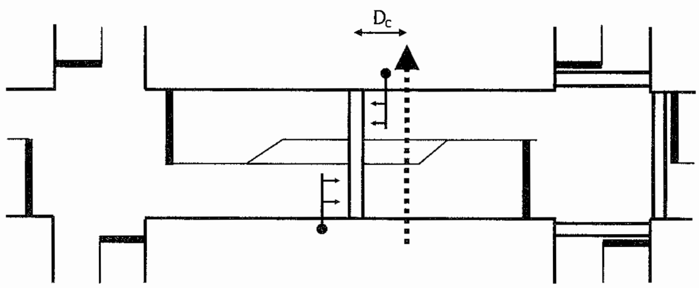
The delay incurred as a consequence of diverting to the nearest signal-controlled crossing is computed first. It includes the delay involved in walking to and from the midsegment crossing point to the nearest signal-controlled crossing and the delay waiting to cross at the signal. Hence, calculation 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 17-19. Exhibit 17-19(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 17-19
Diversion Distance Components



(a) Divert to Nearest Boundary Intersection



(b) Divert to Midsegment Signalized Crosswalk

Exhibit 17-19(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.

The diversion distance to the nearest crossing is computed by using Equation 17-35.

Equation 17-35

$$D_d = 2\,D_c$$

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 17-19(a), then Equation 17-35 applies directly. If the nearest crossing location is at the signalized intersection but the crossing is at Location B, then the distance obtained from Equation 17-35 should be increased by adding two increments of the intersection width W_i.

The delay incurred due to diversion is calculated by using Equation 17-36.

Equation 17-36

$$d_{pd} = \frac{D_d}{S_p} + d_{pc}$$

where

d_{pd} = pedestrian diversion delay (s/p),

D_d = diversion distance (ft),

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

d_{pc} = pedestrian delay when crossing the segment at the nearest signal-controlled crossing (s/p).

The pedestrian delay incurred when crossing at the nearest signal-controlled crossing was determined in Step 3.

B. Compute Roadway Crossing Difficulty Factor

The roadway crossing difficulty factor is computed by using Equation 17-37.

Equation 17-37

$$F_{cd} = 1.0 + \frac{0.10\ d_{px} - (0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606)}{7.5}$$

where

F_{cd} = roadway crossing difficulty factor,

d_{px} = crossing delay = min(d_{pd}, d_{pw}, 60) (s/p),

d_{pd} = pedestrian diversion delay (s/p),

d_{pw} = pedestrian waiting delay (s/p),

$I_{p,link}$ = pedestrian LOS score for link, and

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

If the factor obtained from Equation 17-37 is less than 0.80, then it is set equal to 0.80. If the factor is greater than 1.20, then it is set equal to 1.20.

The pedestrian waiting delay was determined in Step 3. If a midsegment crossing is illegal, then the crossing delay determination does not include consideration of the pedestrian waiting delay d_{pw} [i.e., d_{px} = min(d_{pd}, 60)].

Step 9: Determine Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is computed by using Equation 17-38.

Equation 17-38

$$I_{p,seg} = F_{cd}\left(0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606\right)$$

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

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

17-3 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, then LOS is determined by using Exhibit 17-4 because the pedestrian space concept does not apply.

BICYCLE MODE

This subsection describes the methodology for evaluating the performance of an urban street segment in terms of its service to bicyclists.

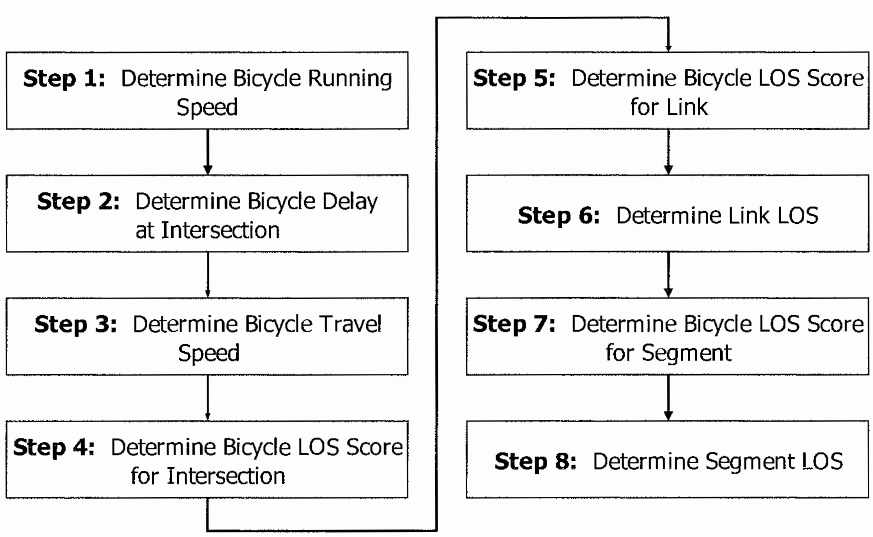
Urban street segment performance from a bicyclist perspective is separately evaluated for each travel direction along the street. *Unless otherwise stated, all variables identified in this section are specific to the subject direction of travel.* The bicycle is assumed to travel in the street (possibly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

The methodology is focused on the analysis of a segment with either signal-controlled or two-way STOP-controlled boundary intersections. Chapter 18 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.

The bicycle methodology is applied through a series of seven steps that culminate in the determination of the segment LOS. These steps are illustrated in Exhibit 17-20. Performance measures that are estimated include bicycle travel speed and LOS scores for the link and segment.

A methodology for evaluating off-street bicycle facilities is provided in Chapter 23, Off-Street Pedestrian and Bicycle Facilities.

Exhibit 17-20
Bicycle Methodology for Urban Street Segments



Link-Based Evaluation

Steps 5 and 6 of the bicycle methodology can be used as a stand-alone procedure 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 fully reflect segment performance.

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, it is recommended that the average running speed of bicycles be taken as 15 mi/h between signalized intersections (*7*). It is recognized that 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), then the delay is equal to 0.0 s/bicycle. If the boundary intersection is signalized, then the delay is computed by using the methodology described in Chapter 18, 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. As such, it is typically slower than the average bicycle running speed. The average bicycle travel speed is computed by using Equation 17-39.

Equation 17-39

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

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, then the bicycle methodology described in Chapter 18 is used for this determination. If the boundary intersection is two-way STOP controlled, then the score is equal to 0.0.

Step 5: Determine Bicycle LOS Score for Link

The bicycle LOS score for the segment $I_{b,link}$ is calculated by using Equation 17-40.

$$I_{b,link} = 0.760 + F_w + F_v + F_S + F_p$$

Equation 17-40

with

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

Equation 17-41

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

Equation 17-42

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

Equation 17-43

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

Equation 17-44

where

$I_{b,link}$ = 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 17-21) (ft),

v_{ma} = adjusted midsegment demand flow rate (see Exhibit 17-21) (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 17-21) (mi/h),

P_{HVa} = adjusted percent heavy vehicles in midsegment demand flow rate (see Exhibit 17-21) (%), and

P_c = pavement condition rating (see Exhibit 17-7).

The value used for several of the variables in Equation 17-41 to Equation 17-44 is dependent on various conditions. These conditions are identified in Column 1 of Exhibit 17-21. If the condition is satisfied, then the equation in Column 2 is used to compute the variable value. If it is not satisfied, then 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 automobile methodology described in a previous subsection.

Step 6: Determine Link LOS

The bicycle LOS for the link is determined by using the bicycle LOS score from Step 5. This performance measure is compared with the thresholds in Exhibit 17-4 to determine the LOS for the specified direction of travel along the subject link.

Exhibit 17-21
Variables for Bicycle LOS Score for Link

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

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_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 7: Determine Bicycle LOS Score for Segment

The bicycle LOS score for the segment is computed by using Equation 17-45.

Equation 17-45

$$I_{b,seg} = 0.160\ I_{b,link} + 0.011\ F_{bi}\ e^{I_{b,int}} + 0.035 \frac{N_{ap,s}}{(L/5280)} + 2.85$$

where

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

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

F_{bi} = indicator variable for boundary intersection control type = 1.0 if signalized, 0.0 if two-way STOP controlled;

$I_{b,int}$ = 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 17-45 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 performance measure is compared with the thresholds in Exhibit 17-4 to determine the LOS for the specified direction of travel along the subject segment.

TRANSIT MODE

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

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.*

The methodology is applicable to public transit vehicles operating in mixed traffic or exclusive lanes and stopping along the street. 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* (3).

The transit methodology is applied through a series of six steps that culminate in the determination of segment LOS. These steps are illustrated in Exhibit 17-22. 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.

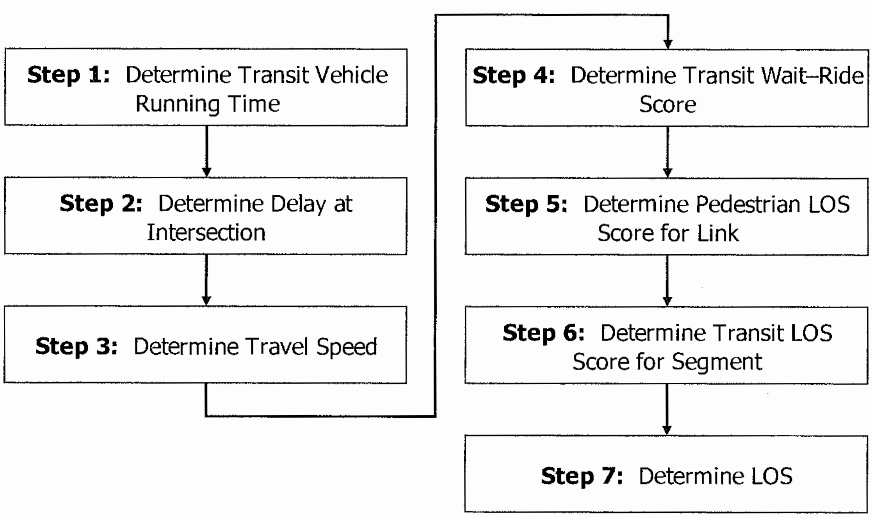


Exhibit 17-22
Transit Methodology for Urban Street Segments

Step 1: Determine Transit Vehicle Running Time

There are two principal components of the transit vehicle's segment running time. One component represents 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 component 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 subpart that describes the calculation of transit vehicle segment running time.

A. Compute Segment Running Speed

Transit vehicle segment running speed represents the speed reached by the vehicle when not influenced by the proximity of a transit stop or traffic control device. This speed can be computed by using Equation 17-46, which is derived from tables given in a Transit Cooperative Research Program report (*8*).

Equation 17-46

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

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 17-6 in Step 2 of the automobile 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 represents the additional time required to decelerate to stop and then accelerate back to the transit vehicle running speed S_{Rt}. It is computed by using Equation 17-47 and Equation 17-48.

Equation 17-47

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

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 17-48

where

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

r_{at} = transit vehicle acceleration rate = 4.0 (ft/s²),

r_{dt} = transit vehicle deceleration rate = 4.0 (ft/s²),

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 17-48 is used to determine the proportion of d_{ad} incurred solely because of a transit stop.

If representative acceleration and deceleration rates are known, then they should be used in Equation 17-47. If these rates are unknown, then a rate of 4.0 ft/s² may be assumed for both acceleration and deceleration (*8*).

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 17-49 is used to compute the delay due to serving passengers.

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

Equation 17-49

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 Input Data subsection of Section 1, Introduction.

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 by using Equation 17-50.

Equation 17-50

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

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

C. Compute Segment Running Time

Equation 17-51 is used to compute transit vehicle running time, which is based on segment running speed and delay due to stops on the segment.

Equation 17-51

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

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

If there are no stops on the segment, then the second term of Equation 17-51 equals zero.

Step 2: Determine Delay at Intersection

The through delay incurred at the boundary intersection is determined in this step. This delay is that incurred by the through movement that exits the segment at the downstream boundary intersection. Guidance for determining this delay is provided in Step 5 of the automobile methodology. Equation 17-52 can be used for a planning analysis to estimate the through delay due to a traffic signal (*8*).

Equation 17-52

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

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 17-52 is obtained from Exhibit 17-23.

Exhibit 17-23
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 (*8*).

Step 3: Determine Travel Speed

Transit travel speed represents an aggregate measure of speed along the street. It combines the delay incurred at the downstream intersection with the segment running time. As such, it is typically slower than the running speed. The transit travel speed is computed by using Equation 17-53.

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

Equation 17-53

where $S_{Tt,seg}$ = travel speed of transit vehicles along the segment (mi/h), t_{Rt} = segment running time of transit vehicle (s), and 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, then 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 subparts 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 17-54.

Equation 17-54

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

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 Input Data subsection.

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 by using Equation 17-55.

Equation 17-55

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

with

Equation 17-56

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

Equation 17-57

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

Equation 17-58

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

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 typical (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 17-56. 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 there are amenities provided at the transit stops.

The first term in Equation 17-56 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 17-56 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 17-56 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, then Equation 17-59 may be used to estimate the average excess wait time.

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

Equation 17-59

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 17-56 represents the amenity time rate reduction. This rate is computed in Equation 17-58 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 (*4*). 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 by using Equation 17-60. A larger score corresponds to better performance.

Equation 17-60

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

where

s_{w-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 $I_{p,link}$ is computed by using the pedestrian methodology, as described in a previous subsection.

Step 6: Determine Transit LOS Score for Segment

The transit LOS score for the segment is computed by using Equation 17-61.

Equation 17-61

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

where $I_{t,seg}$ is the transit LOS score for the segment and 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 17-4 to determine the LOS for the specified direction of travel along the subject street segment.

3. APPLICATIONS

DEFAULT VALUES

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

Exhibit 17-5 and Exhibit 17-6 identify the input data variables associated with the automobile, pedestrian, bicycle, and transit methodologies. These variables can be categorized as either (*a*) suitable for specification as a default value or (*b*) required input data. Those variables categorized as "suitable for specification as a default value" have a minor effect on performance estimates and tend to have a relatively narrow range of typical values used in practice. In contrast, required input variables have either a notable effect on performance estimates or a wide range of possible values. Variables suitable for default value specification are discussed in this subsection.

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

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

Automobile Mode

The required input variables for the automobile methodology are identified in the following list. These variables represent the minimum basic input data that the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 17-5.

- Demand flow rate (at boundary intersection),
- Capacity (at boundary intersection),
- Number of lanes (at boundary intersection),
- Upstream intersection width (at boundary intersection),
- Turn bay length (at boundary intersection),
- Number of through lanes,

- Segment length,
- Restrictive median length (if present),
- Speed limit,
- Through control delay (at boundary intersection),
- Through stopped vehicles (at boundary intersection), and
- Second- and third-term back of queue (at boundary intersection).

Several authoritative reference documents (*9–11*) provide useful guidelines for selecting the type of signal control at the boundary intersection and determining the appropriate traffic control for the segment.

Exhibit 17-24 lists default values for the automobile methodology. Some of the values listed may also be useful for the pedestrian, bicycle, or transit methodologies. The last column of this exhibit indicates "see discussion" for one variable. In this situation, the default value is described in the discussion provided in this subsection.

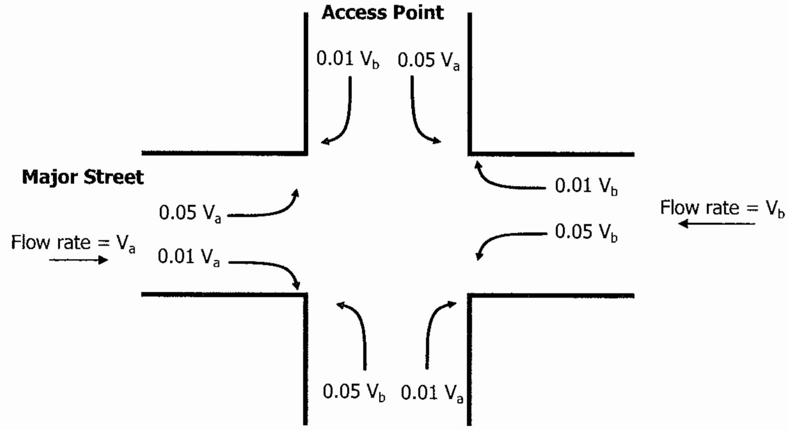
Exhibit 17-24
Default Values: Automobile Mode

Data Category	Input Data Element	Default Values
Traffic characteristics	Access point flow rate	See discussion
	Midsegment flow rate	Estimate by using demand flow rate at the downstream boundary intersection approach
Geometric design	Number of lanes at access points	Segment Approach If median is present, one left-turn lane/approach. If no median is present, no left-turn lanes. No right-turn lanes. Through lanes are the same as N_{th}. Access Point Approach One left-turn lane and one right-turn lane.
	Turn bay length at access points	40% of the access point spacing, where spacing equals 2 (5,280) / D_{ap} in feet. Computed bay length should not exceed 300 ft or be less than 50 ft.
	Proportion of segment with curb	1.0 (curb present on both sides of segment)
	Number of access point approaches	Estimated for each segment side by multiplying default access point density by 1/2 segment length (i.e., $N_{ap,s} = 0.5\ D_{ap}\ L$ / 5,280) Urban arterial D_{ap} = 34 points/mi Suburban arterial D_{ap} = 21 points/mi Urban collector D_{ap} = 61 points/mi Suburban collector D_{ap} = 48 points/mi
Other	Analysis period duration	0.25 h
Performance measures	Midsegment delay	0.0 s/veh
	Midsegment stops	0.0 stops/veh

Note: D_a = access point density on segment (points/mi); $N_{ap,s}$ =number of access point approaches on the right side in the subject direction of travel (points); L = segment length (ft); and N_{th} = number of through lanes on the segment in the subject direction of travel (ln).

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 17-25 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 an access point density consistent with the default densities in Exhibit 17-24 and are applicable to access points serving any public-oriented land use (this excludes single-family residential land use and undeveloped property).

Exhibit 17-25
Default Turn Proportions for Access Point Intersections



If one of the movements shown in Exhibit 17-25 does not exist at a particular access point intersection, then 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 crossing movements at an access point intersection is not needed for the automobile methodology. The left-turn proportions shown are larger than the right-turn proportions because right-turn opportunities are typically more frequent than left-turn opportunities along an arterial street.

Nonautomobile Modes

The required input variables for the pedestrian, bicycle, and transit methodologies are identified in the list below. These variables represent the minimum basic input data that the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 17-6.

<u>Pedestrian Methodology</u>

- Midsegment flow rate
- Pedestrian flow rate
- Downstream intersection width (at boundary intersection)
- Segment length
- Number of through lanes
- Median type and curb presence
- Spacing of objects in buffer
- Legality of midsegment pedestrian crossing
- Proportion of segment adjacent to window display
- Proportion of segment adjacent to building face
- Proportion of segment adjacent to low wall or fence
- Motorized vehicle running speed
- Pedestrian delay

- Pedestrian LOS score for intersection

Bicycle Methodology

- Midsegment flow rate
- Segment length
- Number of through lanes
- Median type and curb presence
- Motorized vehicle running speed
- Bicycle delay (at boundary intersection)
- Bicycle LOS score for intersection

Transit Methodology

- Excess wait time (or on-time performance)
- Transit frequency
- Segment length
- Area type
- Transit stop location
- Transit stop position
- Proportion of stops with shelters
- Proportion of stops with benches
- Motorized vehicle running speed
- Pedestrian LOS score for link
- Through control delay (at boundary intersection)
- Reentry delay
- Effective green-to-cycle-length ratio (at boundary intersection)
- Volume-to-capacity ratio (at roundabout boundary intersection)

Exhibit 17-26 lists the default values for the pedestrian, bicycle, and transit methodologies (*2, 12*).

TYPES OF ANALYSIS

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

Operational Analysis

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

Exhibit 17-26
Default Values: Nonautomobile Modes

Data Category	Input Data Element	Default Value
Traffic characteristics	Dwell time	Downtown stop, transit center, major on-line transfer point, major park-and-ride stop: 60 s Major outlying stop: 30 s Typical outlying stop: 15 s
	Passenger trip length	3.7 mi
	Passenger load factor	0.80 passengers/seat
	Percent heavy vehicles	3%
	Proportion of on-street parking occupied	0.50 (if parking lane present)
Geometric design	Width of outside through lane	12 ft
	Width of bicycle lane	5.0 ft (if provided)
	Width of paved outside shoulder	No parking lane: 1.5 ft (curb and gutter width) Parking lane present: 8.0 ft
	Number of access point approaches	Estimated for each segment side by multiplying default access point density by segment length (i.e., $N_{ap,s} = 0.5\ D_{ap}\ L\ /\ 5{,}280$) Urban arterial D_{ap} = 34 points/mi Suburban arterial D_{ap} = 21 points/mi Urban collector D_{ap} = 61 points/mi Suburban collector D_{ap} = 48 points/mi
	Total walkway width	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
	Effective width of fixed objects	Business or office land use: 2.0 ft inside, 2.0 ft outside Residential or industrial land use: 0.0 ft inside, 0.0 ft outside
	Buffer width	Business or office land use: 0.0 ft Residential or industrial land use: 6.0 ft
Other	Pavement condition rating	3.5
	Distance to nearest signal-controlled crossing	One-third the distance between signal-controlled crossings that bracket the segment
Performance measures	Delay at midsegment signalized crosswalk	20 s/p

Note: D_a = access point density on segment (points/mi); $N_{ap,s}$ =number of access point approaches on the right side in the subject direction of travel (points); L = segment length (ft); and N_{th} = number of through lanes on the segment in the subject direction of travel (ln).

Design Analysis

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

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

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.

Planning and Preliminary Engineering Analysis

The planning and preliminary engineering level of analysis is intended to provide an estimate of the LOS for either a proposed segment or an existing segment in a future year. This level of analysis may also be used 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. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose were described previously in this section.

The requirement for a complete description of the signal timing plan can be a burden for some planning analyses involving signalized intersections, especially when the signal control is pretimed or coordinated–actuated. The intersection Quick Estimation Method 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 Quick Estimation Method 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 section contains specific guidance for the application of alternative tools to the analysis of urban street segments. The tools are described as either simulation or deterministic, in reference to their traffic modeling approach. Additional information on this topic is provided in Volume 4. The focus of this section is the application of alternative tools to evaluate automobile operation.

Strengths of the Automobile Methodology

The automobile methodology described in Section 2 models the driver–vehicle–road system with reasonable accuracy for most applications. It accounts for midsegment speed variations due to traffic and geometric conditions. Alternative tools offer a more detailed treatment of the arrival and departure of vehicles as well as the interaction between the vehicle, the roadway, and the control system. As such, some tools can model the driver–vehicle–road system more accurately for some applications.

The automobile 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 a wide variety of data outputs in a single application. Many alternative tools operate as a "black box," providing little detail describing the intermediate calculations. Moreover, simulation tools require multiple runs and manual calculations to obtain expected values for the output data.

Identified Limitations of the Automobile Methodology

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

Features and Performance Measures Available from Alternative Tools

Both deterministic tools and simulation tools are in common use as alternatives to the procedures offered in this chapter. Deterministic tools are used to a greater extent for the analysis of urban street segments than for most of the other transportation elements represented in this manual. 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.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS assessment for the automobile mode on urban street segments is based on the average travel speed over the segment. The average travel speed is computed by dividing the segment length by the total time required to travel the segment, taking into account all intersection and nonintersection delays.

Alternative tools generally define the 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 Section 2. Therefore, some care must be taken when using speed and delay estimates from other tools. Issues related to speed and delay comparison 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 procedures described in Section 2.

Conceptual Differences That Preclude Direct Comparison of Results

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 in terms of relative magnitude.

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 Chapter 18.

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, it is difficult to achieve comparability of platoon representation along a segment between these tools and the automobile methodology.

Adjustment of Alternative Tool Parameters

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

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

Step-by-Step Recommendations for Applying Alternative Tools

A set of step-by-step recommendations for signalized intersection evaluation with alternative tools is presented in Chapter 18. The recommendations in that chapter also apply to the evaluation of urban street segments.

Sample Calculations Illustrating Alternative Tool Applications

The most useful examples of the application of alternative tools involve multiple segment facilities. Chapter 29, Urban Street Facilities: Supplemental, includes a set of examples to illustrate the use of alternative tools to address the stated limitations of this chapter and Chapter 16, Urban Street Facilities.

Specifically, these examples illustrate (*a*) the application of deterministic tools to optimize signal timing, (*b*) the effect of using a roundabout as a segment boundary, (*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.

4. EXAMPLE PROBLEMS

This part of the chapter describes the application of each of the automobile, pedestrian, bicycle, and transit methodologies through the use of example problems. Exhibit 17-27 provides an overview of these problems. The focus of the examples is 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 17-27
Example Problems

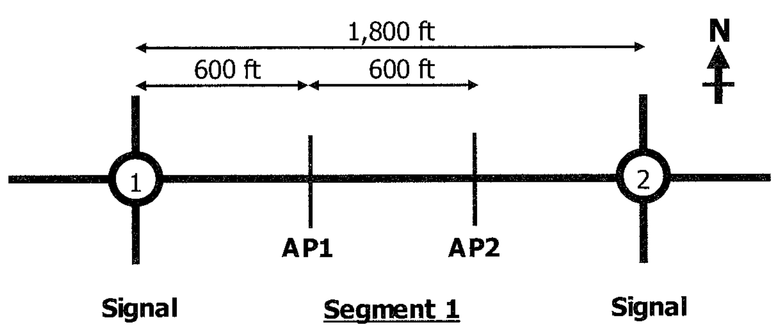
Problem Number	Description	Analysis Level
1	Automobile LOS	Operational
2	Pedestrian LOS	Operational
3	Bicycle LOS	Operational
4	Transit LOS	Operational

EXAMPLE PROBLEM 1: AUTOMOBILE LOS

The Urban Street Segment

The total length of an undivided urban street segment is 1,800 ft. It is shown in Exhibit 17-28. 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 17-28
Example Problem 1: Urban Street Segment Schematic



The segment has two 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." So, the few vehicles that do turn at these locations during the analysis period have been added to the appropriate flow rates at the two 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 17-29. The counts were taken during the 15-min analysis period of interest. However, they have been converted to hourly flow rates. It is noted that the volumes leaving the signalized intersections do not add to equal the volume arriving at the downstream access point intersection.

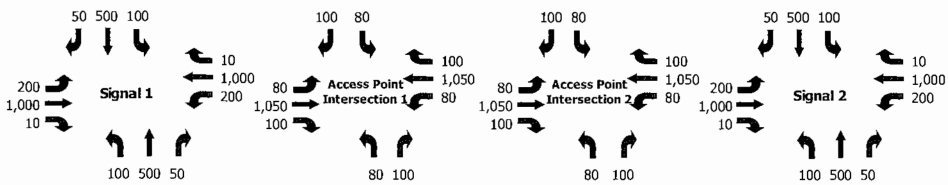


Exhibit 17-29
Example Problem 1: Intersection Turn Movement Counts

The signalization conditions are shown in Exhibit 17-30. 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 17-30
Example Problem 1: Signal Conditions for Intersection 1

Signalized Intersection 1								
General Information								
Cross Street:	First Avenue					**Analysis Period:**	7:15 am to 7:30 am	
Phase Sequence and Left-Turn Mode								
Major street sequence (movement numbers shown)	5 & 1 left leading				Cross street sequence (movement numbers shown)		3 & 7 left leading	
Major street left-turn mode (movement numbers shown)	5/1 Protected-Only				Cross street left-turn mode (movement numbers shown)		3/7 Protected+Permitted	
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
Minimum green, s	5	–	5	–	5	5	5	5
Yellow + red clear, s	3.0	4.0	3.0	4.0	3.0	4.0	3.0	4.0
Phase split, s	20	35	20	35	20	25	20	25
Recall	No	–	No	–	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes
Ref. Phase: 2	Offset, s:	0	Offset Ref.:	End of Green		Force Mode:	Fixed	
	Cycle, s:	100						
Enable Simultaneous Gap-Out? Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑				Enable Dallas Left-Turn Phasing? Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐				

Exhibit 17-30 indicates that the passage time for each phase is 2.0 s. The minimum green setting is 5 s for each phase. 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 coordination is maintained. The cycle length is 100 s.

Geometric conditions and traffic characteristics for Signalized Intersection 1 are shown in Exhibit 17-31. They are the same for Signalized Intersection 2. The movement numbers follow the numbering convention shown in Exhibit 18-2 of Chapter 18.

Exhibit 17-31
Example Problem 1:
Geometric Conditions and Traffic Characteristics for Signalized Intersection 1

Signalized Intersection 1												
Signalized Intersection Input Data *(In each column, enter the volume and lanes data. For all other blue cells, enter values only if there is one or more lanes.)*												
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
Volume, veh/h	200	1,000	10	200	1,000	10	100	500	50	100	500	50
Lanes	1	2	1	1	2	1	1	2	0	1	2	0
Turn bay length, ft	200		200	200		200	200			200		
Sat. flow rate, veh/h/ln	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800		1,800	1,800	
Platoon ratio	1.000	1.333	1.000	1.000	1.333	1.000	1.000	1.000		1.000	1.000	
Initial queue, veh	0	0	0	0	0	0	0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Stop line det. length, ft	40			40			40	40		40	40	
Max. allow. hdwy, s/ve	3.9			3.9			3.9	2.9		3.9	2.9	

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. The saturation flow rate is determined by using the procedure described in Chapter 18.

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 coordinated with the upstream intersection such that favorable progression occurs, as described by a platoon ratio of 1.33. 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 the input platoon ratios shown will only be used for the first iteration of calculations. 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. A 40-ft detection zone is located just upstream of the stop line in each traffic lane at the two signalized intersections.

The geometric conditions that describe the segment are shown in Exhibit 17-32. These data are used to compute the free-flow speed for the segment.

The traffic and lane assignment data for the two access point intersections are shown in Exhibit 17-33. The movement numbers follow the numbering convention shown in Exhibit 19-3 of Chapter 19, Two-Way STOP-Controlled Intersections. There are no turn bays on the segment at the two access point intersections.

Exhibit 17-32
Example Problem 1: Segment Data

Segment 1		
Free-Flow Speed Computation		
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	1,800	1,800
Width of upstream signalized intersection, ft	50	50
Adjusted segment length, ft	1,750	1,750
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	1,750	1,750
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
Access point density, access points/mi	24	24

Exhibit 17-33
Example Problem 1: Access Point Data

Access Point Input Data													
Access Point Location,ft	Approach	**Eastbound**			**Westbound**			**Northbound**			**Southbound**		
	Movement	L	T	R	L	T	R	L	T	R	L	T	R
	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 17-34 provides a summary of the analysis of the individual traffic movements at Signalized Intersection 1.

Exhibit 17-34
Example Problem 1: Movement-Based Output Data

INTERSECTION 1 **Movement:**	**EB L 5**	**EB T 2**	**EB R 12**	**WB L 1**	**WB T 6**	**WB R 16**	**NB L 3**	**NB T 8**	**NB R 18**	**SB L 7**	**SB T 4**	**SB R 14**
Volume, veh/h	200	1,000	10	194	968	10	100	500	50	100	500	50
Lanes	1	2	1	1	2	1	1	2	0	1	2	0
Bay Length, ft	200	0	200	200	0	200	200	0	0	200	0	0
Saturation Flow Rate, veh/h/ln	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800	1,800
Platoon Ratio	1.00	1.33	1.00	1.00	1.33	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	0	0	0	0
Speed Limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
Detector Length, ft	40	0	0	40	0	0	40	40	40	40	40	40
Capacity, veh/h	234	1,703	724	230	1,695	720	213	609	61	213	609	61
Discharge Volume, veh/h	0	1,000	0	0	0	0	0	0	50	100	0	0
Proportion Arriving On Green	0.137	0.630	0.473	0.046	0.456	0.460	0.063	0.189	0.189	0.063	0.189	0.189
Approach Volume, veh/h		1,210			1,172			650			650	
Approach Delay, s/veh		19.4			25.5			39.4			39.4	
Approach Stop Rate, stops/veh		0.471			0.668			0.850			0.850	

With one exception, the first eight rows of data in Exhibit 17-34 are an "echo" of the input data. The remaining rows list variables that are computed by using these input data. The volumes shown in Exhibit 17-34 for the eastbound (EB), northbound (NB), and southbound (SB) movements are identical to the input volumes. The westbound (WB) volumes were reduced from the input volumes during Step 1: Determine Traffic Demand Adjustments. This reduction occurred because the westbound volume input for this intersection exceeded the volume departing the upstream access point intersection (i.e., AP1).

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, then the movement capacity represents the portion of the lane group capacity available to the movement, as distributed in proportion to the flow rate of the movements served by the associated lane group.

Discharge volume is computed for those 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, northbound right-turn movement, and southbound left-turn movement. A value of 0.0 veh/h is shown for all other movements and indicates that they are not relevant to this calculation. If volume exceeds capacity for any given movement, then 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 17-34 represent summary statistics for the approach. The approach volume represents 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 17-35 provides a summary of the output data for Signalized Intersection 1 using a signal controller perspective. The controller has eight timing functions (or timers), with Timers 1 to 4 representing Ring 1 and Timers 5 to 8 representing Ring 2. The ring structure and phase assignments are described in Chapter 18. 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 17-35
Example Problem 1: Timer-Based Phase 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**
Assigned Phase	1	2	3	4	5	6	7	8
Case No	2	3	1	4	2	3	1	4
Phase Duration (G+Y+Rc), s	16.48	51.29	9.32	22.90	16.69	51.09	9.32	22.90
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	36.22	52.70	4.00	13.32	36.22	52.92	4.00	13.32
Phase End Time, s	52.70	4.00	13.32	36.22	52.92	4.00	13.32	36.22
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	29.78	0.00	17.00	31.68	29.78	0.00	17.00	31.68
Max. Queue Clearance Time (g_c+I1),	13.238	0.000	6.644	16.955	13.432	0.000	6.644	16.955
Green Extension Time (g_e), s	0.302	0.000	0.098	1.946	0.313	0.000	0.098	1.946
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	0.023	0.000	0.000	0.000	0.023

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 to Phase 2), coupled with the occasional need for lagging left-turn phases and split phasing, creates the situation in which *phases* do not always time in sequence. For example, with a lagging left-turn phase sequence, major-street through Phase 2 times first and then major-street left-turn Phase 1 times second.

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

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

The 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 case number shown in Exhibit 17-35 is used as a single variable descriptor of each possible combination of left-turn mode and lane group type (i.e., shared or exclusive). An understanding of this variable is not needed to interpret the output data.

The phase duration shown in Exhibit 17-35 represents the estimated average phase duration during the analysis period. It represents the sum of the green, yellow change, and red clearance intervals. For Timer 2 (i.e., Phase 2), the average green interval duration can be computed as 47.29 s (= 51.29 – 4.00).

The phase start time represents the time the timer (and phase) starts, relative to system time 0.0. For Phase 2, the start time is 52.70 s. The end of the green interval associated with this phase is 100.0 s (= 52.70 + 47.29). 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 represents 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 17-35 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 Chapter 18.

Timer-Based Movement Data

Exhibit 17-36 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 17-34) assigned to each timer.

The saturation flow rate shown in Exhibit 17-36 represents the saturation flow rate for the movement. The procedure for calculating these rates is described in Chapter 18. 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 17-36
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	1,710.00		1,710.00		1,710.00		1,710.00	
Through Movement Data								
Assigned Movement		2		4		6		8
Mvmt. Sat Flow, veh/h		3,600.00		3,222.18		3,600.00		3,222.18
Right-Turn Movement Data								
Assigned Movement		12		14		16		18
Mvmt. Sat Flow, veh/h		1,530.00		321.15		1,530.00		321.15

Timer-Based Lane Group Data

The methodology described in Chapter 18 computes a variety of output data that describe the operation of each intersection lane group. The example problem in Chapter 18 illustrates these data 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 segment through movements.

Access Point Data

Exhibit 17-37 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 17-37
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	0.170			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	1,200											

The eastbound and westbound volumes listed in Exhibit 17-37 are not equal to the input volumes. These volumes were adjusted during Step 1: Determine Traffic Demand Adjustments such 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., 0.17) with that for the northbound (minor-street) left turn (i.e., 0.26) 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 influence 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 0.10, indicating that the inside lane is blocked about 10% of the time.

Results

Exhibit 17-38 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 represent the movements exiting the segment at a boundary intersection. Thus, the westbound movements on Segment 1 are those that occur at Signalized Intersection 1. Similarly, the eastbound movements on Segment 1 are those that occur at Signalized Intersection 2.

Exhibit 17-38
Example Problem 1: Performance Measure Summary

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		33.48			33.48	
1	Running Speed, mph		36.65			36.65	
1	Through Delay, s/veh		20.862			20.862	
1	Travel Speed, mph		22.58			22.58	
1	Stop Rate, stops/veh		0.608			0.608	
1	Spatial Stop Rate, stops/mi		1.78			1.78	
1	Through vol/cap ratio		0.57			0.57	
1	Percent of Base FFS		55.4			55.4	
1	Level of Service		C			C	
1	Proportion Left Lanes		0.33			0.33	
1	Auto. Traveler Perception Score		2.56			2.56	
SPILLBACK TIME, h: never							

The spillback check procedure computes the time of spillback for each of the internal movements. For turn movements, the bay/lane spillback time represents the time before the turn bay overflows. For through movements, the bay/lane spillback time represents the time before the through lane overflows due only to through demand. If a turn bay exists and it overflows, then 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, then the time of first spillback is reported at the bottom of Exhibit 17-38.

The output data for the two through movements are listed in Exhibit 17-38, 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 represents 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 percent of base free-flow speed equals the travel speed divided by the base free-flow speed. It and the through movement volume-to-capacity ratio are used with Exhibit 17-2 to determine that the segment is operating at LOS C in both travel directions.

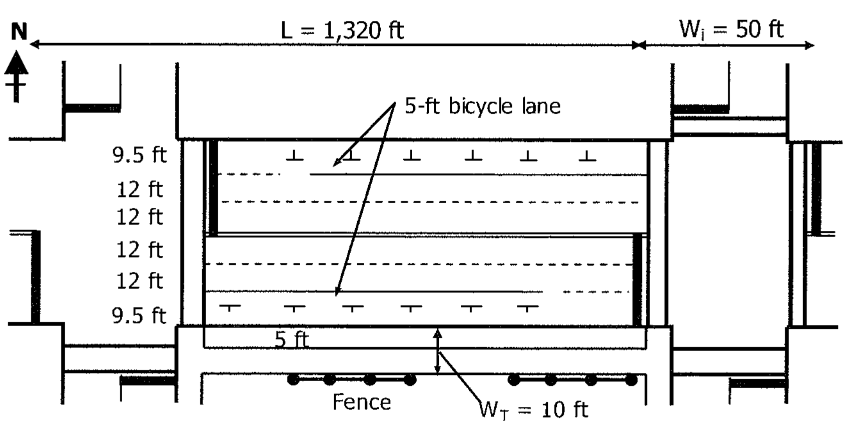
Each travel direction has one left-turn bay and three intersections. Thus, the proportion of intersections with left-turn lanes is 0.33. This proportion is used in Step 10: Determine Automobile Traveler Perception Score to compute the score of 2.56, which suggests that most automobile travelers would 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 17-39. 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 17-39
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 17-39. Both boundary intersections are signalized. It is legal to cross the segment at uncontrolled, midsegment locations. The following additional information is known about the sidewalk and street segment:

<u>Traffic Characteristics</u>

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

<u>Geometric Characteristics</u>

Shoulder width consists of 8.0 ft for parking and 1.5 ft for gutter pan.

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

<u>Other Data</u>

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

<u>Performance Measures Obtained from Supporting Methodologies</u>

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 waiting delay: 740 s/p

Pedestrian LOS score for the downstream intersection: 3.6

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 17-16. Next, the pedestrian travel speed along the sidewalk will be calculated. Finally, LOS for the crossing will be determined by using the computed pedestrian LOS score and pedestrian space variables with Exhibit 17-3.

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 by using Equation 17-23.

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

$$W_{s,i} = \max(5.0, 1.5)$$

$$W_{s,i} = 5.0 \text{ ft}$$

The shy distance on the outside of the sidewalk is computed by using Equation 17-24.

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

$$W_{s,o} = 3.0(0.0) + 2.0\ (0.0) + 1.5\ (0.50)$$

$$W_{s,o} = 0.75 \text{ ft}$$

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 by using Equation 17-22.

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

$$W_E = 10 - 0.0 - 0.0 - 5.0 - 0.75$$

$$W_E = 4.25 \text{ ft}$$

The pedestrian flow per unit width of sidewalk is computed by using Equation 17-27 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 by using Equation 17-28.

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

$$S_p = (1 - 0.00078\ [7.84]^2)\ 4.4$$

$$S_p = 4.19 \text{ ft/s}$$

Finally, Equation 17-29 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 17-16 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 18, Signalized Intersections, was used to estimate two pedestrian delay values. One value is the pedestrian delay at the boundary intersection when walking parallel to segment d_{pp}. This delay was computed to be 40 s/p. The second value is the pedestrian delay when 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 19, Two-Way STOP-Controlled Intersections, was used to estimate the delay incurred while waiting for an acceptable gap in traffic d_{pw}. This delay was computed to be 740 s/p.

Step 4: Determine Pedestrian Travel Speed

The pedestrian travel speed is computed by using Equation 17-30.

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

$$S_{Tp,seg} = \frac{1{,}320}{\frac{1{,}320}{4.19} + 40}$$

$$S_{Tp,seg} = 3.72 \text{ ft/s}$$

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 18 was used to determine the pedestrian LOS score for the downstream boundary intersection $I_{p,int}$. 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 total width of the outside through lane, bicycle lane, and paved shoulder W_t is computed as

$$W_t = W_{ol} + W_{bl}$$

$$W_t = 12 + 5$$

$$W_t = 17 \text{ ft}$$

In fact, the variable W_t does not include the width of the paved outside shoulder in this instance because the proportion of occupied on-street parking exceeds 0.0.

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 because the midsegment flow rate is greater than 160 veh/h.

The street cross section is curbed, so the adjusted width of paved outside shoulder W_{os}^* is 8.0 ft (= 9.5 – 1.5).

Because the proportion of occupied on-street parking is less than 0.25, the effective width of the combined bicycle lane and shoulder W_1 is computed as

$$W_1 = W_{bl} + W_{os}^*$$

$$W_1 = 5 + 8$$

$$W_1 = 13 \text{ ft}$$

The adjusted available sidewalk width W_{aA} is computed as

$$W_{aA} = \min(W_T - W_{buf}, 10)$$

$$W_{aA} = \min(10 - 5, 10)$$

$$W_{aA} = 5.0 \text{ ft}$$

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 automobile methodology described in Section 2 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 by using Equation 17-32.

$$F_w = -1.2276 \ln(W_v + 0.5\, W_1 + 50\, p_{pk} + W_{buf}\, f_b + W_{aA}\, f_{sw})$$

$$F_w = -1.2276 \ln(17 + 0.5\,(13) + 50\,(0.20) + 5.0\,(1.0) + 5.0\,(4.5))$$

$$F_w = -5.05$$

The motorized vehicle volume adjustment factor is computed by using Equation 17-33.

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

$$F_v = 0.0091 \frac{940}{4\,(2)}$$

$$F_v = 1.07$$

The motorized vehicle speed adjustment factor is computed by using Equation 17-34.

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

$$F_s = 4\left(\frac{33.0}{100}\right)^2$$

$$F_s = 0.44$$

Finally, the pedestrian LOS score for the link $I_{p,link}$ is calculated by using Equation 17-31.

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

$$I_{p,link} = 6.0468 + (-5.05) + 1.07 + 0.44$$

$$I_{p,link} = 2.51$$

Step 7: Determine Link LOS

The pedestrian LOS for the link is determined by using the pedestrian LOS score from Step 6 and the average pedestrian space from Step 2. These two performance measures are compared with their respective thresholds in Exhibit 17-3 to determine that the LOS for the specified direction of travel along the subject link is C.

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 17-36.

$$d_{pd} = \frac{D_d}{S_p} + d_{pc}$$

$$d_{pd} = \frac{880}{4.19} + 80$$

$$d_{pd} = 290 \text{ s/p}$$

The crossing delay used to estimate the roadway crossing difficulty factor is computed as

$$d_{px} = \min(d_{pd}, d_{pw}, 60)$$

$$d_{px} = \min(290, 740, 60)$$

$$d_{px} = 60 \text{ s/p}$$

The roadway crossing difficulty factor is computed by using Equation 17-37.

$$F_{cd} = 1.0 + \frac{0.10\ d_{px} - (0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606)}{7.5} \leq 1.20$$

$$F_{cd} = 1.0 + \frac{0.10\ (60) - (0.318\ [2.51] + 0.220\ [3.60] + 1.606)}{7.5}$$

$$F_{cd} = 1.20$$

Step 9: Determine Pedestrian LOS Score for Segment

The pedestrian LOS score for the segment is computed by using Equation 17-38.

$$I_{p,seg} = F_{cd}\left(0.318\ I_{p,link} + 0.220\ I_{p,int} + 1.606\right)$$

$$I_{p,seg} = 1.20\ \left(0.318\ [2.51] + 0.220[3.60] + 1.606\right)$$

$$I_{p,seg} = 3.83$$

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 17-3 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 17-40.

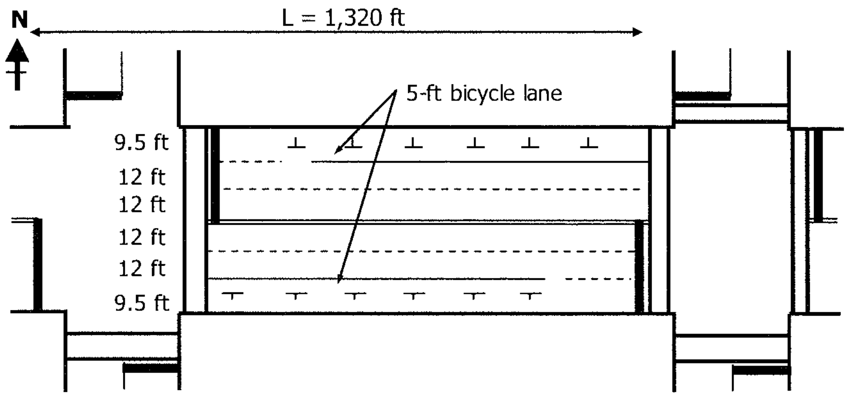


Exhibit 17-40
Example Problem 3: Segment Geometry

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 17-40. 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

Shoulder width consists of 8.0 ft for parking and 1.5 ft for gutter pan.

Cross section has raised curb along outside edge of 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 17-4.

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 methodology in Chapter 18, 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 by using Equation 17-39.

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

$$S_{Tb,seg} = \frac{3{,}600\ (1{,}320)}{5{,}280\ (60.0 + 40.0)}$$

$$S_{Tb,seg} = 9.0\text{ mi/h}$$

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 18 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 total width of the outside through lane, bicycle lane, and paved shoulder W_t is computed as

$$W_t = W_{ol} + W_{bl}$$

$$W_t = 12 + 5$$

$$W_t = 17 \text{ ft}$$

In fact, the variable W_t does not include the width of the paved outside shoulder in this instance because the proportion of occupied on-street parking exceeds 0.0.

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 because the midsegment flow rate is greater than 160 veh/h.

The street cross section is curbed, so the adjusted width of paved outside shoulder W_{os}^* is 8.0 ft (= 9.5 – 1.5).

Because the combined bicycle lane and adjusted shoulder width exceed 4.0 ft, the effective width of the outside through lane is computed as

$$W_e = W_v + W_{bl} + W_{os}^* - 20\, p_{pk} \quad \geq 0.0$$

$$W_e = 17 + 5 + 8 - 20\,(0.20)$$

$$W_e = 26 \text{ ft}$$

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 automobile methodology described in Section 2 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. This speed exceeds 21 mi/h, so 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 by using Equation 17-41.

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

$$F_w = -0.005\,(26)^2$$

$$F_w = -3.38$$

The motorized vehicle volume adjustment factor comes from Equation 17-42.

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

$$F_v = 0.507 \ln\left(\frac{940}{4\,(2)}\right)$$

$$F_v = 2.42$$

The motorized vehicle speed adjustment factor is computed by using Equation 17-43.

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

$$F_S = 0.199\left[1.1199\ln(33.0 - 20) + 0.8103\right](1 + 0.1038\ (8.0))^2$$

$$F_S = 2.46$$

The pavement condition adjustment factor is computed by using Equation 17-44.

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

$$F_p = \frac{7.066}{2.0^2}$$

$$F_p = 1.77$$

Finally, the bicycle LOS score for the link $I_{b,link}$ is calculated by using Equation 17-40.

$$I_{b,link} = 0.760 + F_w + F_v + F_S + F_p$$

$$I_{b,link} = 0.760 - 3.38 + 2.42 + 2.46 + 1.77$$

$$I_{b,link} = 4.02$$

Step 6: Determine Link LOS

The bicycle LOS for the link is determined by using the bicycle LOS score from Step 5. This performance measure is compared with the thresholds in Exhibit 17-4 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 bicycle LOS score for the segment is computed by using Equation 17-45.

$$I_{b,seg} = 0.160\ I_{b,link} + 0.011\ F_{bi}\ e^{I_{b,int}} + 0.035\frac{N_{ap,s}}{(L/5{,}280)} + 2.85$$

$$I_{b,seg} = 0.160\ (4.02) + 0.011\ (1)\ e^{0.080} + 0.035\frac{3}{1{,}320/5{,}280} + 2.85$$

$$I_{b,seg} = 3.92$$

Step 8: Determine Segment LOS

The bicycle LOS for the segment is determined by using the bicycle LOS score from Step 7. This performance measure is compared with the thresholds in Exhibit 17-4 to determine that the LOS for the specified direction of travel along the subject segment is D.

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 17-41. A bus stop is provided on the south side of the segment for the subject route.

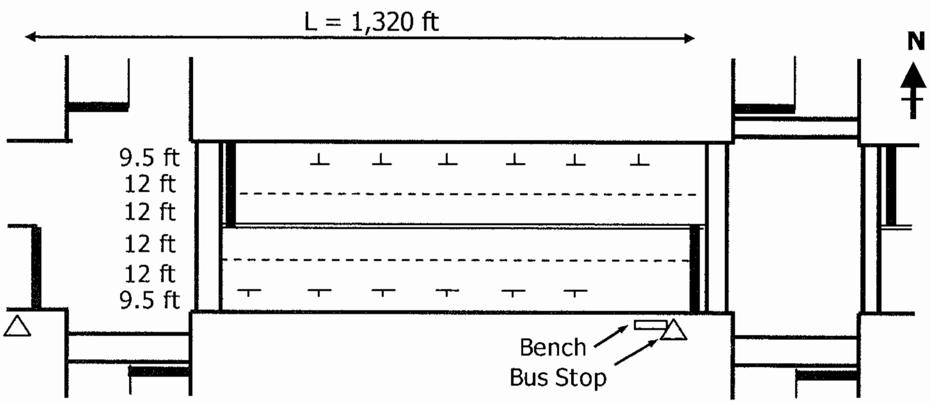


Exhibit 17-41
Example Problem 4: Segment Geometry

The Question

What is the transit LOS for the eastbound bus route while traveling the subject segment?

The Facts

The geometric details of the segment are shown in Exhibit 17-41. 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:

<u>Transit Characteristics</u>

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

<u>Other Data</u>

Area type: not in a central business district

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 control delay at downstream boundary intersection: 20.9 s/veh

Reentry delay: 16.17 s

g/C ratio at downstream boundary intersection: 0.4729

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 17-4.

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 by using Equation 17-46.

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

$$S_{Rt} = \min\left(33.0, \frac{61}{1+e^{-1.00+(1{,}185\,[1]/1{,}320)}}\right)$$

$$S_{Rt} = 32.1\text{ mi/h}$$

The acceleration and deceleration rates are unknown, so they are assumed to equal 4.0 ft/s².

The bus stop is located on the near side of a signalized intersection. From Equation 17-48, 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 47.29 s (calculated as the phase duration minus the change period), and the cycle length is 100 s. Therefore, f_{ad} is 0.4729.

Equation 17-47 can now be used to compute the portion of bus stop delay due to acceleration and deceleration.

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

$$d_{ad} = \frac{5{,}280}{3{,}600}\left(\frac{33.0}{2}\right)\left(\frac{1}{4.0}+\frac{1}{4.0}\right)(0.4729)$$

$$d_{ad} = 5.56 \text{ s}$$

Equation 17-49 is used to compute the portion of bus stop delay due to serving passengers, using the input average dwell time of 20.0 s and an f_{dt} value of 0.4729, based on 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 upon 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. By following the procedures given in Chapter 18, Signalized Intersections, this time is calculated to be 16.17 s.

Equation 17-50 is used to compute the total delay due to the transit stop.

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

$$d_{ts} = 5.56 + 9.46 + 16.17 = 31.19 \text{ s}$$

Equation 17-51 is used to compute transit vehicle running time on the basis of the previously computed components.

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

$$t_{Rt} = \frac{3{,}600\ (1{,}320)}{5{,}280\ (32.1)} + 31.19$$

$$t_{Rt} = 59.3 \text{ s}$$

Step 2: Determine Delay at Intersection

The automobile control delay d at the boundary intersection was computed to be 20.9 s/veh.

Step 3: Determine Travel Speed

The average transit travel speed is computed by using Equation 17-53.

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

$$S_{Tt,seg} = \frac{3{,}600\ (1{,}320)}{5{,}280\ (59.3 + 20.9)}$$

$$S_{Tt,seg} = 11.2 \text{ mi/h}$$

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 by using Equation 17-54.

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

$$F_h = 4.00\ e^{-1.434/(4 + 0.001)}$$

$$F_h = 2.80$$

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 17-58. A default passenger trip length of 3.7 mi is used in the absence of other information.

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

$$T_{at} = \frac{1.3\ (0.0) + 0.2\ (1.0)}{3.7} = 0.054 \text{ min/mi}$$

Since no information is available for actual excess wait time, but on-time performance information is available for the route, Equation 17-59 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 by using Equation 17-57.

$$a_1 = 1 + \frac{(4)(F_l - 0.80)}{4.2}$$

$$a_1 = 1 + \frac{(4)(0.83 - 0.80)}{4.2}$$

$$a_1 = 1.03$$

The perceived travel time rate is computed by using Equation 17-56.

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

$$T_{ptt} = \left(1.03 \frac{60}{11.2} \right) + \left(2\, [0.043] \right) - 0.054$$

$$T_{ptt} = 5.53 \text{ min/mi}$$

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 by using Equation 17-55.

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

$$F_{tt} = \frac{(-0.40-1)\,(4.0) - (-0.40+1)(5.53)}{(-0.40-1)\,(5.53) - (-0.40+1)\,(4.0)}$$

$$F_{tt} = 0.88$$

Finally, the transit wait–ride score is computed by using Equation 17-60.

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

$$s_{w-r} = (2.80)(0.88)$$

$$s_{w-r} = 2.46$$

Step 5: Determine Pedestrian LOS Score for Link

The pedestrian methodology described in Section 2 was used to determine the pedestrian LOS score for the link $I_{p,link}$. 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 by using Equation 17-61.

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

$$I_{t,seg} = 6.0 - 1.50\,(2.46) + 0.15\,(3.53)$$

$$I_{t,seg} = 2.84$$

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 17-4 to determine that the LOS for the specified bus route is C.

5. REFERENCES

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CHAPTER 18
SIGNALIZED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

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

OVERVIEW OF THE METHODOLOGY

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

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

Analysis Boundaries

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

Analysis Level

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

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

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

Study Period and Analysis Period

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

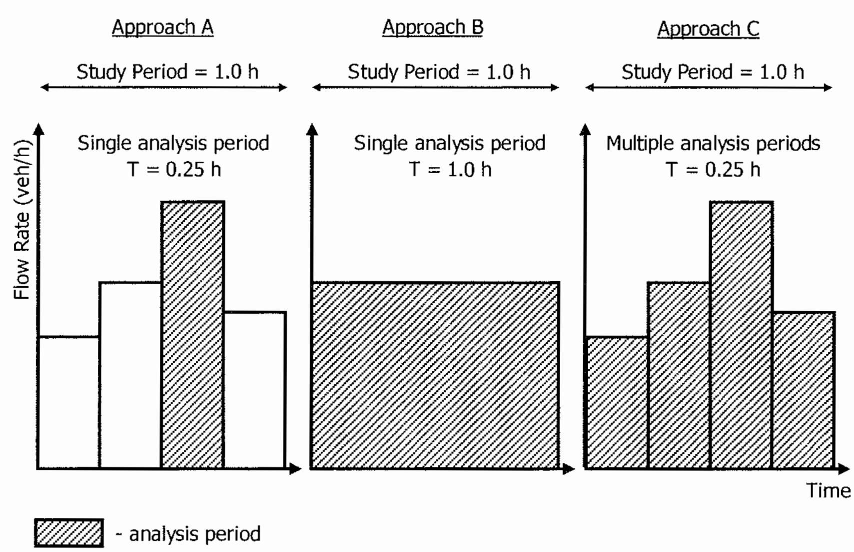
The methodology is based on the assumption that traffic conditions are steady during the analysis period (i.e., systematic change over time is negligible). For this reason, the analysis period ranges from 0.25 to 1 h. The longer durations are sometimes used for planning analyses. In general, the analyst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

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

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

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

Exhibit 18-1
Three Alternative Study Approaches



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

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

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

Performance Measures

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

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

Travel Modes

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

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

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

Lane Groups and Movement Groups

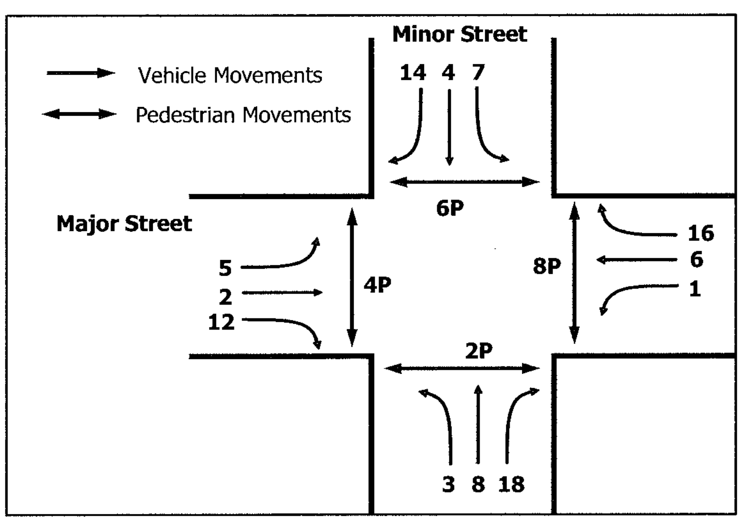
The automobile methodology is designed to evaluate the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group*. In general, a separate lane group is established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

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

Movement and Phase Numbering

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

Exhibit 18-2
Intersection Traffic Movements and Numbering Scheme

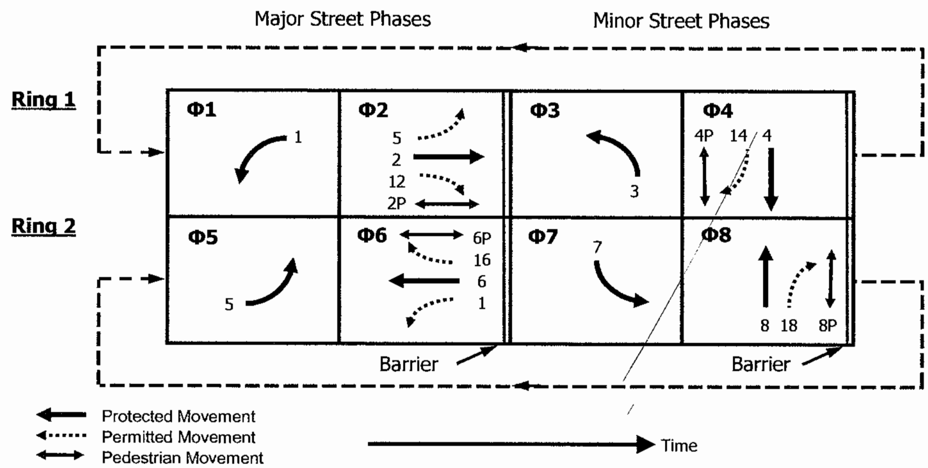


Modern actuated controllers implement signal phasing by using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other. The commonly used eight-phase dual-ring structure is shown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

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

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

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



LOS CRITERIA

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

Automobile Mode

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

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

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

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

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

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

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

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

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

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

Exhibit 18-4
LOS Criteria: Automobile Mode

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

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

Nonautomobile Modes

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

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

Exhibit 18-5
LOS Criteria: Pedestrian and Bicycle Modes

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

REQUIRED INPUT DATA

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

Automobile Mode

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

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

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

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

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

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

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

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

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

Traffic Characteristics Data

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

Demand Flow Rate

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

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

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

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

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

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

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

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

Right-Turn-on-Red Flow Rate

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

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

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

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

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

Percent Heavy Vehicles

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

Intersection Peak Hour Factor

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

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

Equation 18-1

where

PHF = peak hour factor,

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

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

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

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

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

Platoon Ratio

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

Exhibit 18-8 Relationship Between Arrival Type and Progression Quality

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

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

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

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

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

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

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

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

Equation 18-2

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

where

R_p = platoon ratio,

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

g = effective green time (s), and

C = cycle length (s).

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

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

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

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

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

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

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

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

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

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

Upstream Filtering Adjustment Factor

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

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

Equation 18-3

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

where

I = upstream filtering adjustment factor, and

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

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

Initial Queue

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

Base Saturation Flow Rate

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

Lane Utilization Adjustment Factor

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

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

Equation 18-4

where

- f_{LU} = adjustment factor for lane utilization,
- v_g = demand flow rate for movement group (veh/h),
- v_{g1} = demand flow rate in the single exclusive lane with the highest flow rate of all exclusive lanes in movement group (veh/h/ln), and
- N_e = number of exclusive lanes in movement group (ln).

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

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

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

Pedestrian Flow Rate

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

Bicycle Flow Rate

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

On-Street Parking Maneuver Rate

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

Local Bus Stopping Rate

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

Geometric Design Data

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

Number of Lanes

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

Average Lane Width

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

Number of Receiving Lanes

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

Turn Bay Length

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

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

Presence of On-Street Parking

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

Approach Grade

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

Signal Control Data

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

Type of Signal Control

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

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

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

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

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

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

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

Phase Sequence

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

Left-Turn Operational Mode

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

Dallas Left-Turn Phasing Option

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

Passage Time

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

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

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

Maximum Green

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

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

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

Minimum Green

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

Yellow Change and Red Clearance

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

Walk

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

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

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

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

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

Pedestrian Clear

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

Phase Recall

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

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

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

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

Dual Entry

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

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

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

Simultaneous Gap-Out

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

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

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

Cycle Length (Coordinated-Actuated Operation)

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

Phase Splits (Coordinated-Actuated Operation)

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

Offset and Offset Reference Point (Coordinated-Actuated Operation)

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

Force Mode (Coordinated-Actuated Operation)

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

Other Data

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

Analysis Period Duration

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

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

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

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

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

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

Speed Limit

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

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

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

Stop-Line Detector Length and Detection Mode

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

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

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

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

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

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

Area Type

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

Nonautomobile Modes

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

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

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

Exhibit 18-9
Input Data Requirements: Nonautomobile Modes

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

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

Traffic Characteristics Data

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

Permitted Left-Turn Flow Rate

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

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

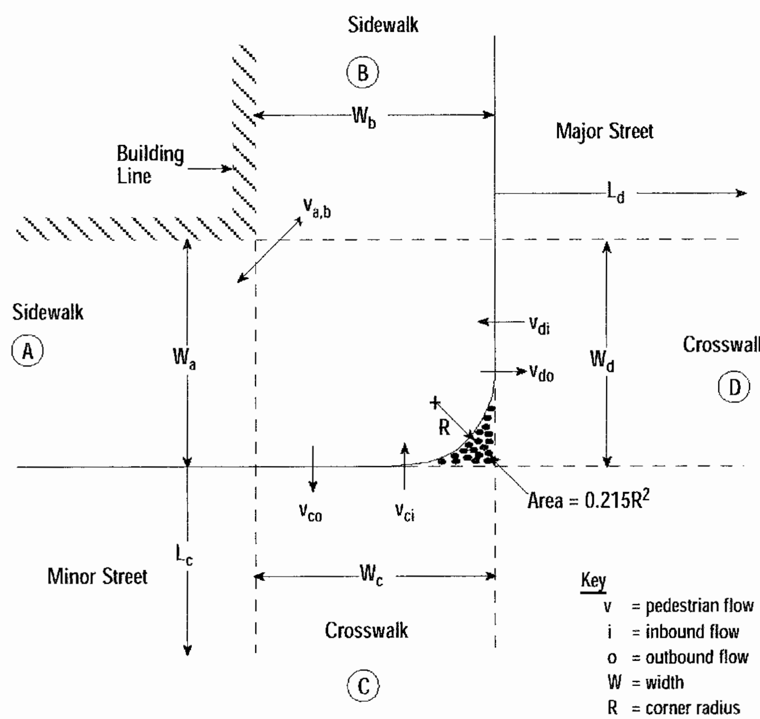
Midsegment 85th Percentile Speed

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

Pedestrian Flow Rate

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

Exhibit 18-10
Intersection Corner Geometry and Pedestrian Movements



Proportion of On-Street Parking Occupied

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

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

Geometric Design Data

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

Street Width

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

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

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

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

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

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

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

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

Signal Control Data

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

Rest in Walk

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

Cycle Length

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

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

Pedestrian Signal Head Presence

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

Duration of Phase Serving Pedestrians and Bicycles

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

SCOPE OF THE METHODOLOGY

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

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

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

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

LIMITATIONS OF THE METHODOLOGY

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

Automobile Mode

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

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

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

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

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

Nonautomobile Modes

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

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

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

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

2. METHODOLOGY

OVERVIEW

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

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

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

AUTOMOBILE MODE

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

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

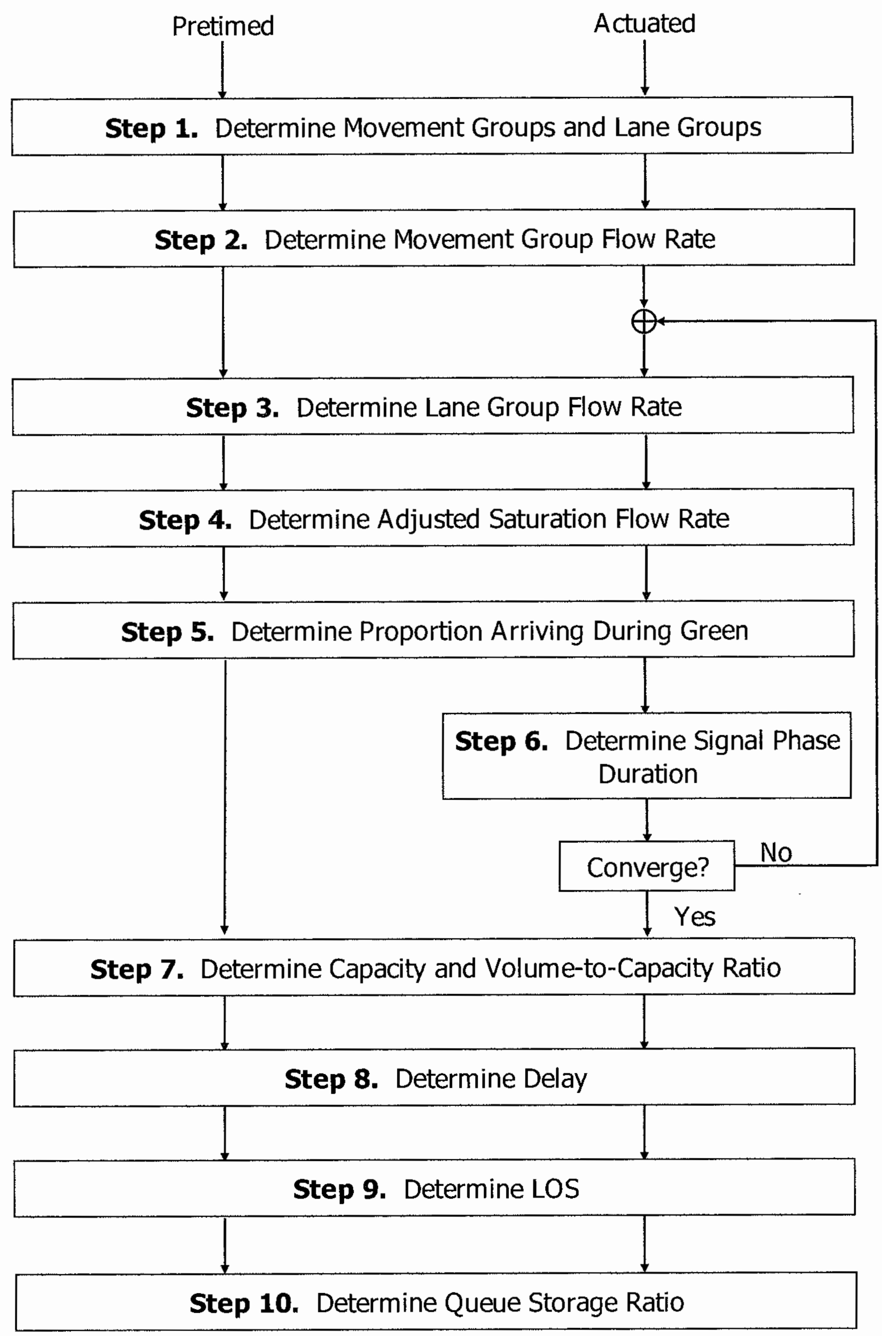
Because of the intensity of the computations, the objective of this subsection is to introduce the analyst to the calculation process and discuss the key analytic procedures. This objective is achieved by focusing the discussion on lane groups that serve one traffic movement with pretimed control and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

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

Framework

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

Exhibit 18-11
Automobile Methodology for Signalized Intersections



Step 1: Determine Movement Groups and Lane Groups

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

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

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

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

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

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

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

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

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

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

Exhibit 18-12
Typical Lane Groups for Analysis

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

Step 2: Determine Movement Group Flow Rate

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

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

Step 3: Determine Lane Group Flow Rate

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

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

Step 4: Determine Adjusted Saturation Flow Rate

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

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

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

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

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

Equation 18-5

where

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

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

f_w = adjustment factor for lane width,

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

f_g = adjustment factor for approach grade,

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

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

f_a = adjustment factor for area type,

f_{LU} = adjustment factor for lane utilization,

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

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

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

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

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

Base Saturation Flow Rate

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

Adjustment for Lane Width

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

Exhibit 18-13
Lane Width Adjustment Factor

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

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

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

Adjustment for Heavy Vehicles

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

Equation 18-6

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

where

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

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

Adjustment for Grade

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

Equation 18-7

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

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

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

Adjustment for Parking

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

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

Equation 18-8

where

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

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

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

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

Adjustment for Bus Blockage

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

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

Equation 18-9

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

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

Adjustment for Area Type

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

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

Adjustment for Lane Utilization

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

Adjustment for Right Turns

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

Equation 18-10

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

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

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

Adjustment for Left Turns

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

Equation 18-11

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

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

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

Adjustment for Pedestrians and Bicycles

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

Step 5: Determine Proportion Arriving During Green

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

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

Equation 18-12

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

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

Step 6: Determine Signal Phase Duration

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

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

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

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

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

Equation 18-13

where

D_p = phase duration (s),

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

g_s = queue service time (s),

g_e = green extension time (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

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

Exhibit 18-14
Time Elements Influencing Actuated Phase Duration

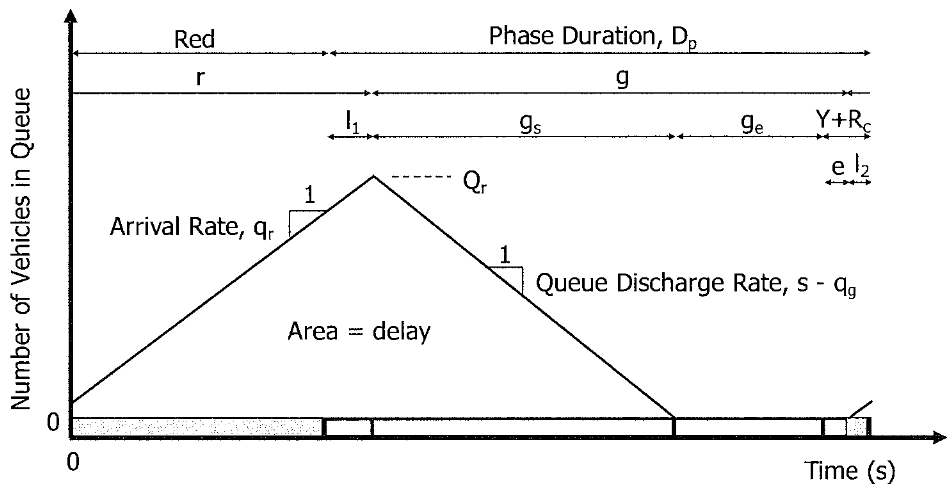


Exhibit 18-14 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the red interval ends. At this time, the queue begins to discharge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

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

Equation 18-14

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

where

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

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

all other variables are as previously defined.

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

Lane Group Volume-to-Capacity Ratio

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

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

Equation 18-15

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

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

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

Equation 18-16

where

X = volume-to-capacity ratio,

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

c = capacity (veh/h).

Critical Intersection Volume-to-Capacity Ratio

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

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

Equation 18-17

with

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

Equation 18-18

where

X_c = critical intersection volume-to-capacity ratio,

C = cycle length (s),

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

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

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

L = cycle lost time (s).

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

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

Identifying Critical Lane Groups and Critical Flow Ratios

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

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

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

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

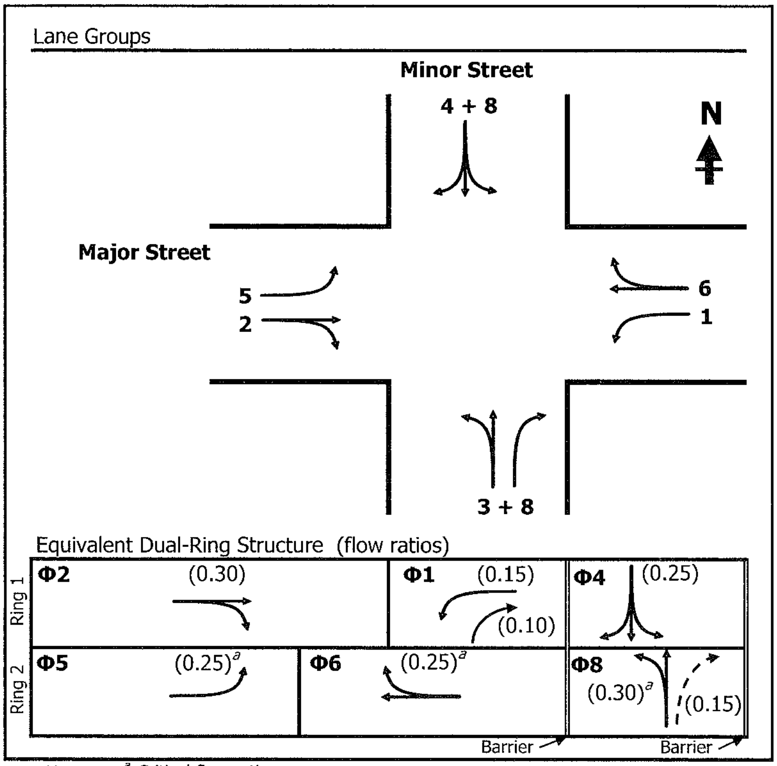
Finally, the process is repeated for the phases between the other barrier pair. One critical flow rate is defined for each barrier pair by this process. These two values are then added to obtain the sum of the critical flow ratios used in Equation 18-17. The lost time associated with each of the critical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

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

Basic Case

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

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



Note: [a] Critical flow ratio.

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

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

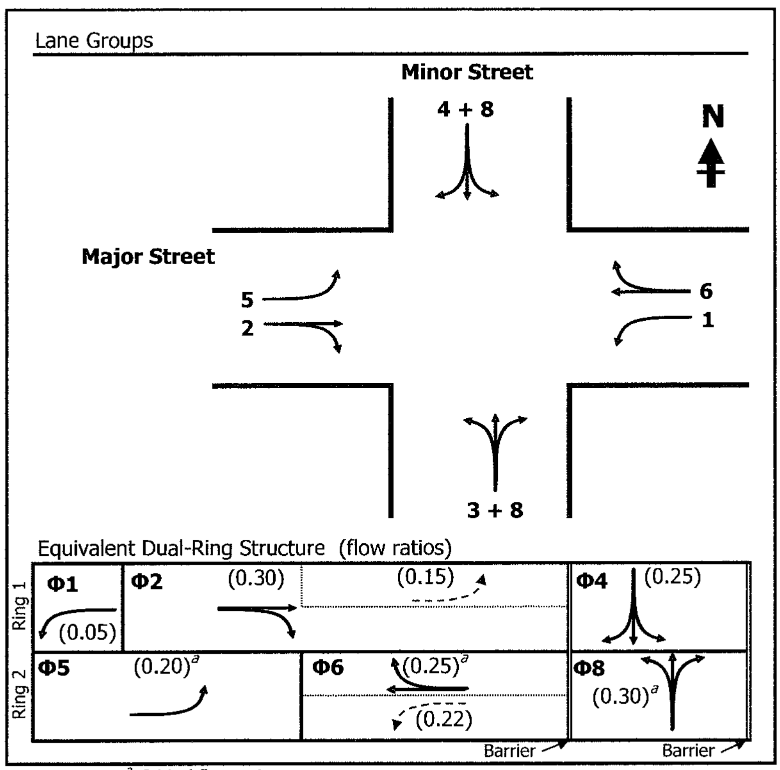
There are two possible critical paths through the major-street phase sequence—one path is associated with Phases 1 and 2 (i.e., Ring 1), and the other path is associated with Phases 5 and 6 (i.e., Ring 2). The total phase flow ratio for the Ring 1 path is 0.30 + 0.15, or 0.45. The total phase flow ratio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

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

Special Case: Pretimed Protected-Permitted Left-Turn Operation

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

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



Note: [a] Critical flow ratio.

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

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

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

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

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

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

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

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

Step 8: Determine Delay

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

Equation 18-19

$$d = d_1 + d_2 + d_3$$

where

- d = control delay (s/veh),
- d_1 = uniform delay (s/veh),
- d_2 = incremental delay (s/veh), and
- d_3 = initial queue delay (s/veh).

Concepts

Uniform Delay

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

Equation 18-20

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

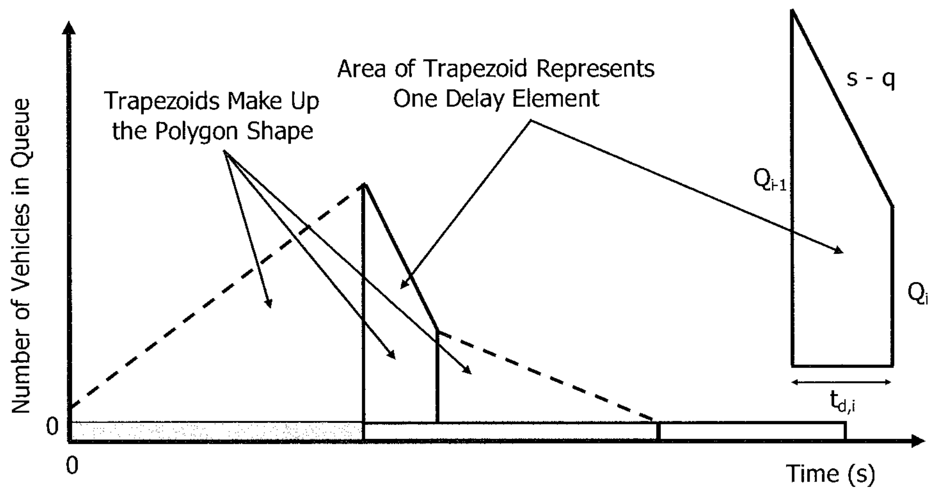
All variables are as previously defined. The delay calculation procedure used in this methodology is consistent with Equation 18-20. However, it removes the aforementioned assumptions to allow more accurate uniform delay estimates for progressed traffic movements, movements with multiple green periods,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

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

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

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

Exhibit 18-17
Decomposition of Queue Accumulation Polygon



Equation 18-21

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

where

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

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

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

all other variables as previously defined.

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

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

Equation 18-22

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

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

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

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

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

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

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

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

Equation 18-23

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

and

Equation 18-24

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

where

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

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

all other variables as previously defined.

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

Incremental Delay

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

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

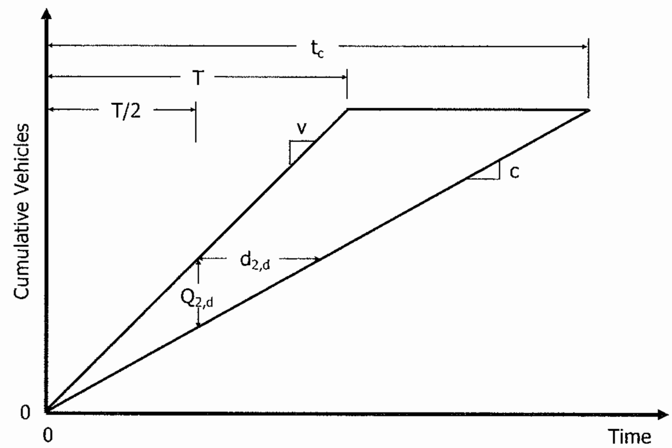


Exhibit 18-18 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate v during analysis period T, which has capacity c. The deterministic delay component is represented by the triangular area bounded by the thick line and is associated with an average delay per vehicle represented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

Initial Queue Delay

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

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

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

Exhibit 18-19
Initial Queue Delay with Increasing Queue Size

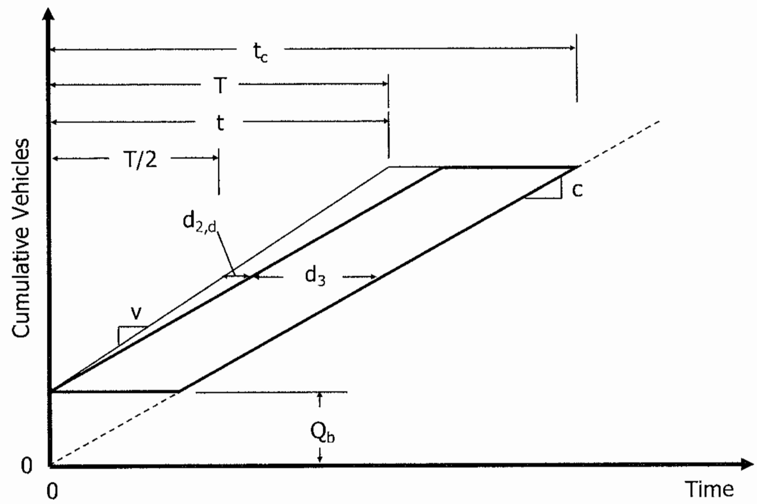


Exhibit 18-20
Initial Queue Delay with Decreasing Queue Size

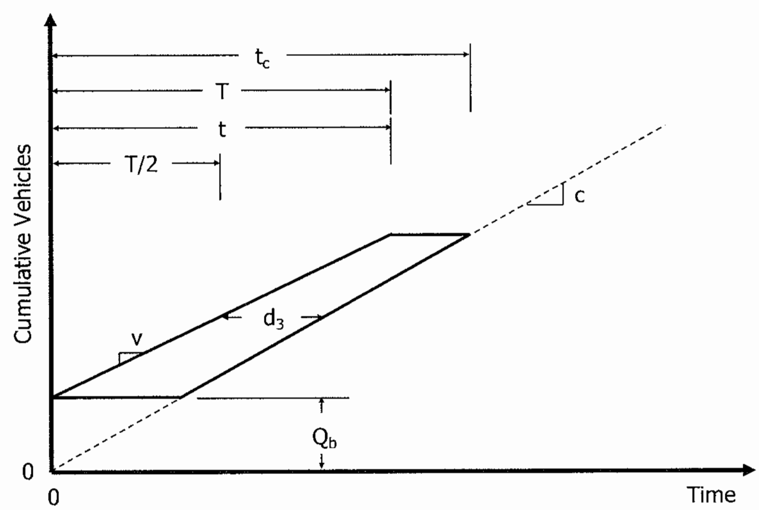
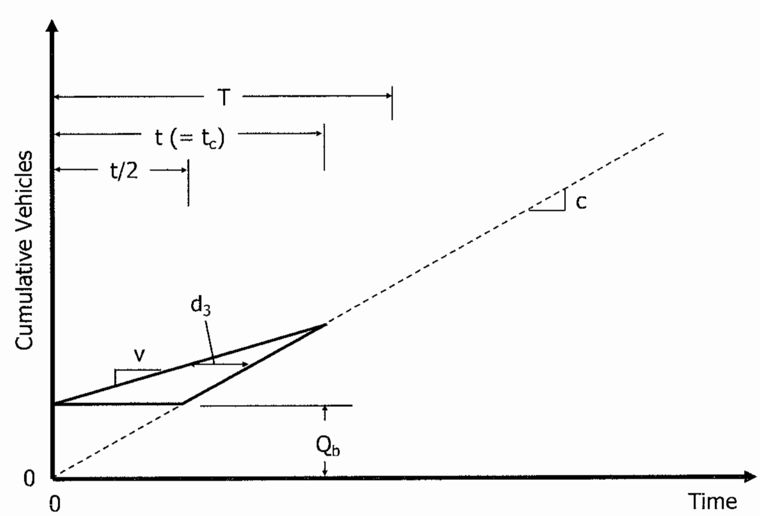


Exhibit 18-21
Initial Queue Delay with Queue Clearing



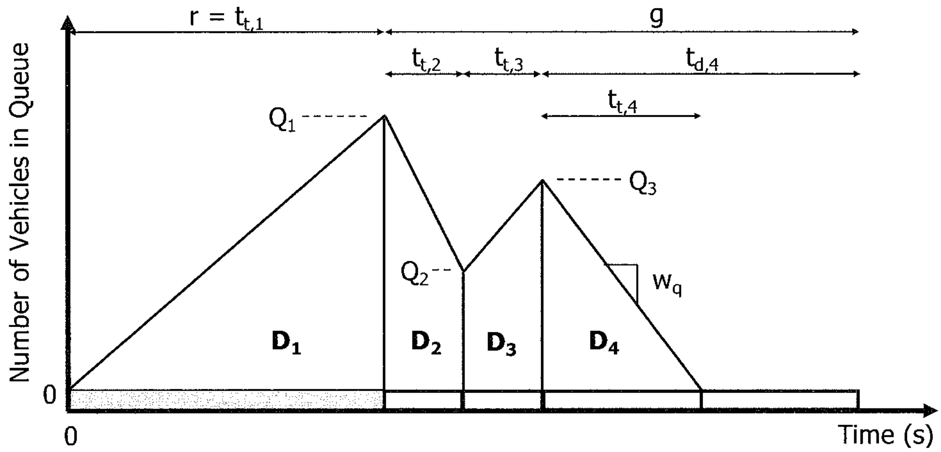
In this chapter, the phrase *initial queue* is always used in reference to the initial queue due to unmet demand in the previous time period. It *never* refers to vehicles in queue due to random, cycle-by-cycle fluctuations in demand.

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

A. Compute Baseline Uniform Delay

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

Exhibit 18-22
Polygon for Uniform Delay Calculation



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

Equation 18-25

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

with

Equation 18-26

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

where

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

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

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

all other variables as previously defined.

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

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

B. Initial Queue Analysis

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

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

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

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

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

If $v \geq c_s$, then

Equation 18-27

$$t = T$$

If $v < c_s$, then

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

Equation 18-28

where

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

T = analysis period duration (h),

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

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

c_s = saturated capacity (veh/h).

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

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

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

Equation 18-29

where

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

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

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

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

C. Compute Uniform Delay

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

If lane group i has an initial queue, then

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

Equation 18-30

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

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

Equation 18-31

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

D. Compute Average Capacity

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

If lane group i has an initial queue, then

Equation 18-32

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

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

Equation 18-33

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

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

E. Compute Initial Queue Delay

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

Equation 18-34

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

with

Equation 18-35

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

If $v \geq c_A$, then

Equation 18-36

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

Equation 18-37

$$t_A = T$$

If $v < c_A$, then

Equation 18-38

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

Equation 18-39

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

where

- t_A = adjusted duration of unmet demand in the analysis period (h),
- Q_e = queue at the end of the analysis period (veh),
- Q_{eo} = queue at the end of the analysis period when $v \geq c_A$ and $Q_b = 0.0$ (veh), and

other variables as previously defined.

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

$$t_c = t_A + Q_e / c_A$$ Equation 18-40

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

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

F. Compute Incremental Delay Factor

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

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

$$k = (1 - 2k_{min})(v / c_a - 0.5) + k_{min} \quad \le 0.50$$ Equation 18-41

with

$$k_{min} = -0.375 + 0.354\,PT - 0.0910\,PT^2 + 0.00889\,PT^3 \quad \ge 0.04$$ Equation 18-42

$$c_a = 3{,}600\,\frac{g_a\, s\, N}{C}$$ Equation 18-43

$$g_a = G_{max} + Y + R_c - l_1 - l_2$$ Equation 18-44

where

k = incremental delay factor,

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

k_{min} = minimum incremental delay factor, and

g_a = available effective green time (s).

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

G. Compute Incremental Delay

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

Equation 18-45

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

with

Equation 18-46

$$X_A = v / c_A$$

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

H. Compute Lane Group Control Delay

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

I. Compute Aggregated Delay Estimates

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

Equation 18-47

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

where

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

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

m_j = number of lane groups on approach j.

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

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

Equation 18-48

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

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

Step 9. Determine LOS

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

Step 10. Determine Queue Storage Ratio

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

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

Extension to Multiple Time Periods

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

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

Interpretation of Results

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

Level of Service

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

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

Volume-to-Capacity Ratio

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

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

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

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

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

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

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

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

Volume-to-Capacity Ratio and Delay Combinations

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

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

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

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

PEDESTRIAN MODE

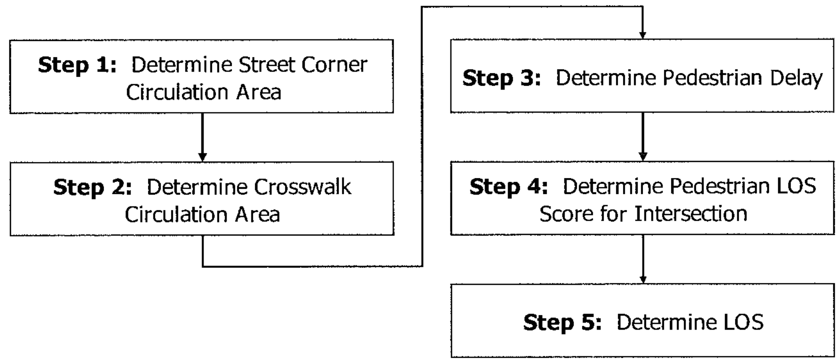
This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to pedestrians.

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

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

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

Exhibit 18-23
Pedestrian Methodology for Signalized Intersections



Concepts

Performance Measures

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

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

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

Exhibit 18-24
Qualitative Description of Pedestrian Space

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

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

Flow Conditions

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

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

Exhibit 18-25
Condition 1: Minor-Street Crossing

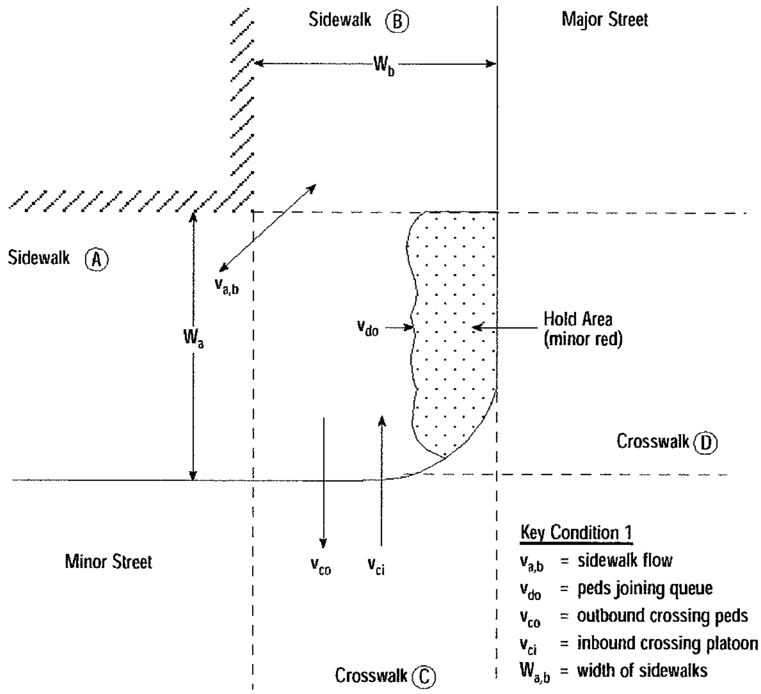
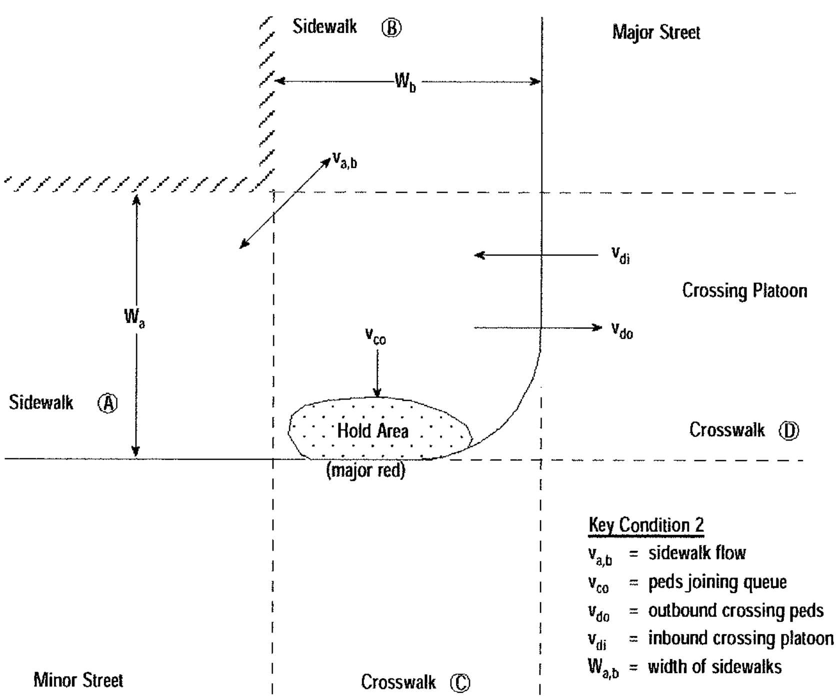


Exhibit 18-26
Condition 2: Major-Street Crossing



Effective Walk Time

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

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

Equation 18-49

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

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

Equation 18-50

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

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

Equation 18-51

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

where

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

$Walk$ = pedestrian walk setting (s),

PC = pedestrian clear setting (s),

D_p = phase duration (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

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

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

Step 1: Determine Street Corner Circulation Area

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

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

A. Compute Available Time–Space

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

Equation 18-52

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

where

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

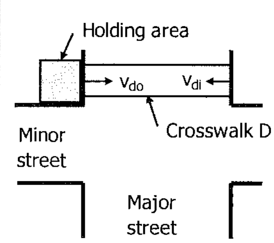
C = cycle length (s),

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

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

R = radius of corner curb (ft).

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



B. Compute Holding-Area Waiting Time

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

Equation 18-53

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

with

Equation 18-54

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

where

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

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

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

C = cycle length (s), and

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

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

Equation 18-55

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

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

Equation 18-56

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

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

Equation 18-57

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

where

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

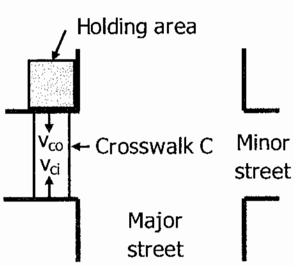
$Walk_{mi}$ = pedestrian walk setting for the phase serving the minor-street through movement (s),

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

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

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

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



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

C. Compute Circulation Time–Space

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

Equation 18-58

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

where TS_c is the time–space available for circulating pedestrians (ft²-s) and other variables are as previously defined.

D. Compute Pedestrian Corner Circulation Area

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

Equation 18-59

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

with

Equation 18-60

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

where

M_{corner} = corner circulation area per pedestrian (ft²/p),

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

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

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

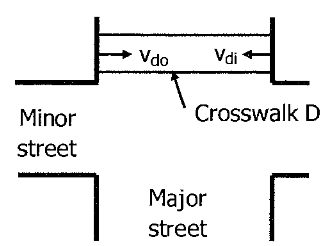
v_{di} = flow rate of pedestrians arriving at the corner after crossing the major street (p/h), and

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

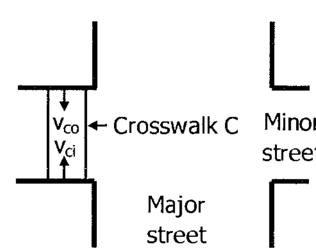
Other variables are as previously defined. The circulation area obtained from Equation 18-59 can be compared with the ranges provided in Exhibit 18-24 to make some judgments about the performance of the subject intersection corner.

Step 2: Determine Crosswalk Circulation Area

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



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



A. Establish Walking Speed

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

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

B. Compute Available Time–Space

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

Equation 18-61

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

where

TS_{cw} = available crosswalk time–space (ft^2-s),

L_d = length of Crosswalk D (ft),

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

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

C. Compute Effective Available Time–Space

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

Equation 18-62

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

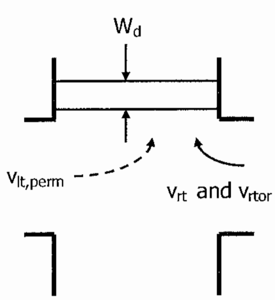
with

Equation 18-63

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

Equation 18-64

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



where

$TS_{cw}{}^{*}$ = effective available crosswalk time–space (ft^2-s),

TS_{tv} = time–space occupied by turning vehicles (ft^2-s),

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

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

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

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

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

D. Compute Pedestrian Service Time

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

If crosswalk width W_d is greater than 10 ft, then

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

Equation 18-65

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

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

Equation 18-66

with

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

Equation 18-67

where

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

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

other variables are as previously defined.

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

E. Compute Crosswalk Occupancy Time

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

Equation 18-68

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

with

Equation 18-69

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

where

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

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

Other variables are as previously defined.

F. Compute Pedestrian Crosswalk Circulation Area

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

Equation 18-70

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

where M_{cw} is the crosswalk circulation area per pedestrian (ft²/p) and other variables are as previously defined.

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

Step 3: Determine Pedestrian Delay

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

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

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

Equation 18-71

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

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

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

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

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

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

Step 4: Determine Pedestrian LOS Score for Intersection

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

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

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

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

Equation 18-72

with

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

Equation 18-73

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

Equation 18-74

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

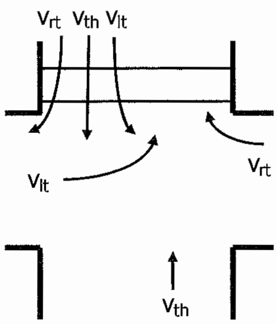
Equation 18-75

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

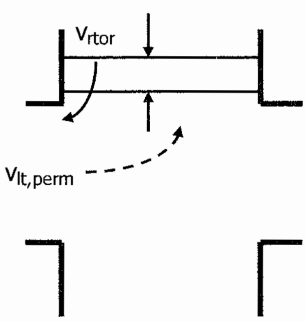
Equation 18-76

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

Equation 18-77



Movements crossing Crosswalk D



where

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

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_{delay} = pedestrian delay adjustment factor,

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

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

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

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

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

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

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

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

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

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

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

Step 5: Determine LOS

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

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

BICYCLE MODE

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

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

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

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

Exhibit 18-27
Bicycle Methodology for Signalized Intersections

Step 1: Determine Bicycle Delay

↓

Step 2: Determine Bicycle LOS Score for Intersection

↓

Step 3: Determine LOS

Step 1: Determine Bicycle Delay

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

A. Compute Bicycle Lane Capacity

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

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

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

Equation 18-78

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

where

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

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

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

C = cycle length (s).

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

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 18-79.

Equation 18-79

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

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

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

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

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

Step 2: Determine Bicycle LOS Score for Intersection

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

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

Equation 18-80

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

with

Equation 18-81

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

Equation 18-82

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

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

Equation 18-83

where

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Step 3: Determine LOS

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

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

3. APPLICATIONS

DEFAULT VALUES

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

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

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

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

Automobile Mode

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

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

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

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

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

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

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

Platoon Ratio

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

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

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

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

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

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

Exhibit 18-29
Progression Quality and Arrival Type

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

Lane Utilization Adjustment Factor

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

Exhibit 18-30
Default Lane Utilization Adjustment Factors

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

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

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

On-Street Parking Maneuver Rate

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

Exhibit 18-31
Default Parking Maneuver Rate

Street Type	Number of Spaces in 250 ft	Parking Time Limit (h)	Turnover Rate (veh/h)	Maneuver Rate (maneuvers/h)
Two-way	10	1	1.0	16
		2	0.5	8
One-way	20	1	1.0	32
		2	0.5	16

Automobile Mode (Coordinated-Actuated Operation)

Exhibit 18-32 lists the default values for evaluating signalized intersections that are part of a coordinated-actuated signal system. The text "see discussion" in the last column of this exhibit indicates that the default value is described in the discussion provided in this part.

Exhibit 18-32
Default Values: Automobile Mode with Coordinated-Actuated Signal Control

Data Category	Input Data Element	Default Value
Controller settings	Cycle length	See discussion
	Phase splits	See discussion
	Offset	Equal to travel time in Phase 2 direction [a]
	Offset reference	End of green for Phase 2 [a]
	Force mode	Fixed

Note: [a] Assumes that Phase 2 is the reference phase. Substitute 6 if Phase 6 is the reference phase.

Cycle Length

The cycle length used for a coordinated signal system often represents a compromise value based on intersection capacity, queue size, phase sequence, segment length, speed, and progression quality. Consideration of these factors leads to the default cycle lengths shown in Exhibit 18-33.

Exhibit 18-33
Default System Cycle Length

Average Segment Length (ft)[a]	Cycle Length by Street Class and Left-Turn Phasing (s)[b]					
	Major Arterial Street			Minor Arterial Street or Grid Network		
	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets
1,300	90	120	150	60	80	120
2,600	90	120	150	100	100	120
3,900	110	120	150			

Notes: [a] Average length based on all street segments in the signal system.
[b] Selected left-turn phasing column should describe the phase sequence at the high-volume intersections in the system.

Phase Splits

If the phase splits are not known, they can be estimated by using the quick estimation method described in Chapter 31. The method can be used to estimate the effective green time for each phase on the basis of the established system cycle length. The phase split D_p is then computed by adding 4 s of lost time to the estimated effective green time (i.e., $D_p = g + 4.0$).

Nonautomobile Modes

The required input variables for the pedestrian and bicycle methodologies are identified in the following list. These variables represent the minimum basic input data the analyst will need to provide for an analysis. These variables were previously defined in the text associated with Exhibit 18-9.

- Demand flow rate of motorized vehicles,
- RTOR flow rate (pedestrian mode only),
- Permitted left-turn flow rate (pedestrian mode only),
- Pedestrian flow rate (pedestrian mode only),
- Bicycle flow rate (bicycle mode only),
- Number of lanes,
- Crosswalk length (pedestrian mode only), and
- Pedestrian signal head presence (pedestrian mode only).

The RTOR flow rate does not have a default value for application of the pedestrian methodology. This flow rate has both a notable effect on performance estimates and a wide range of possible values. The analyst is encouraged to conduct measurements at intersections for the purpose of developing local defaults for this variable.

The permitted left-turn flow rate for movements served by the permitted mode is equal to the left-turn demand flow rate. The permitted left-turn flow rate for movements served by the protected-permitted mode does not have a default value. This flow rate has both a notable effect on performance estimates and a wide range of possible values. It should be measured in the field if possible. If the analysis is dealing with future conditions or if the permitted left-turn flow rate is not known from field data, its value can be approximated as the left-turn arrival rate during the permitted period of the protected-permitted operation. This rate should equal the left-turn arrival rate during the effective red time [i.e., $q_r = (1 - P)qC/r$].

The pedestrian flow rate data consist of count data for each of five pedestrian movements at each intersection corner. These variables are shown as $v_{a,b}$, v_{ci}, v_{co}, v_{di}, and v_{do} in Exhibit 18-10.

Exhibit 18-34 lists the default values for the pedestrian and bicycle methodologies (*25–27*).

TYPES OF ANALYSIS

The automobile, pedestrian, and bicycle methodologies described in this chapter can each be used in three types (or levels) of analysis. These analysis levels are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis level are described in later parts of this subsection.

Operational Analysis

Each of the methodologies is most easily applied at an operational level of analysis. At this level, all traffic, geometric, and signalization conditions are specified as input variables by the analyst. These input variables are used in the methodology to compute various performance measures.

Exhibit 18-34
Default Values: Nonautomobile Modes

Data Category	Input Data Element	Default Value
Traffic characteristics	Intersection peak hour factor (motorized vehicles)	If analysis period is 0.25 h and hourly data are used: Total entering volume ≥ 1,000 veh/h: 0.92 Total entering volume < 1,000 veh/h: 0.90 Otherwise: 1.00
	Midsegment 85th percentile speed	Speed limit (mi/h)
	Proportion of on-street parking occupied	0.50 (if parking lane present)
Geometric design	Street width	Based on a 12-ft lane width
	Number of right-turn islands	None
	Width of outside through lane	12 ft
	Width of bicycle lane	5.0 ft (if provided)
	Width of paved outside shoulder	No parking lane: 2.0 ft (curb and gutter width) Parking lane present: 8.0 ft
	Total walkway width	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
	Crosswalk width	12 ft
	Corner radius	Trucks and buses in turn volume: 45 ft No trucks or buses in turn volume: 25 ft
Signal control	Walk	Actuated: 7 s Pretimed: green interval minus pedestrian clear
	Pedestrian clear	Based on 3.5-ft/s walking speed
	Rest in walk	Not enabled
	Cycle length	Based on default values determined for automobile mode
	Yellow change + red clearance [a]	4 s
	Duration of phases serving pedestrians and bicycles	Based on default values determined for automobile mode
Other	Analysis period duration	0.25 h

Note: [a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

Design Analysis

The design level of analysis has two variations. Both variations require specifying (*a*) traffic conditions and (*b*) target levels for a specified set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

The second variation of the design level requires the additional specification of geometric conditions. The methodology is then applied by using an iterative approach in which alternative signalization conditions are separately evaluated.

The objective with either variation is to identify alternatives that operate at the target level of the specified performance measures (or provide a better level of performance). The analyst may then recommend the "best" alternative based on consideration of the full range of factors.

Planning and Preliminary Engineering Analysis

The planning and preliminary engineering level of analysis is intended to provide an estimate of the LOS for either a proposed intersection or an existing intersection in a future year. This level of analysis may also be used for a preliminary engineering activity to size the overall geometrics of a proposed intersection.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses. Therefore, default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose were described previously in this section.

The requirement for a complete description of the signal timing plan can be a burden for some planning analyses, especially when the signal control is pretimed or coordinated-actuated. The quick estimation method described in Chapter 31 can be used to estimate a reasonable timing plan, in conjunction with the aforementioned default values.

USE OF ALTERNATIVE TOOLS

General alternative tool guidance is provided in Chapters 6 and 7.

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for applying alternative tools to the analysis of signalized intersections. Additional information on this topic may be found in the Technical Reference Library in Volume 4.

Strengths of the Automobile Methodology

The automobile methodology described in Section 2 offers a comprehensive procedure for analyzing the performance of a signalized intersection. It models the driver–vehicle–road–signal system with reasonable accuracy for most applications. Simulation-based traffic analysis tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with the roadway and the control system. As such, some simulation tools can model the driver–vehicle–road–signal system more accurately for some applications.

The automobile methodology offers the following advantages over the use of simulation-based analysis tools:

- Its empirically calibrated saturation flow rate adjustment factors can produce an accurate estimate of saturation flow rate (simulation tools require saturation flow rate as an input variable).
- It produces a direct estimate of capacity and volume-to-capacity ratio (these measures are much more difficult to quantify with simulation).
- It produces an estimate of expected, long-run performance for a variety of measures (multiple runs and supplemental calculations are required to obtain this type of estimate with a simulation tool).

Identified Limitations of the Automobile Methodology

The limitations of the automobile methodology are identified near the end of Section 1. If any of these limitations applies to a particular situation, then alternative tools may produce more credible performance estimates. Limitations involving consideration of the impact of progression on performance are a special case that is discussed in more detail in Chapter 16, Urban Street Facilities.

Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the capacity, control delay, LOS, and back of queue associated with a lane group at a signalized intersection. Alternative tools often offer additional performance measures such as number of stops, fuel consumption, air quality, and operating costs.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS assessment for signalized intersections is based on control delay, which is defined as the excess travel time caused by the action of the control device (in this case, the signal).

Simulation-based analysis tools often use a definition of delay that is different from that used in the automobile methodology, especially for movements that are oversaturated at some point during the analysis. Therefore, some care must be taken in the determination of LOS when simulation-based delay estimates are used. Delay comparison among different tools is discussed in more detail in Chapter 7.

An accurate estimate of control delay may be obtained from a simulation tool by performing simulation runs with and without the control device(s) in place. The segment delay reported with no control is the delay due to geometrics and interaction between vehicles. The additional delay reported in the run with the control in place is, by definition, the control delay.

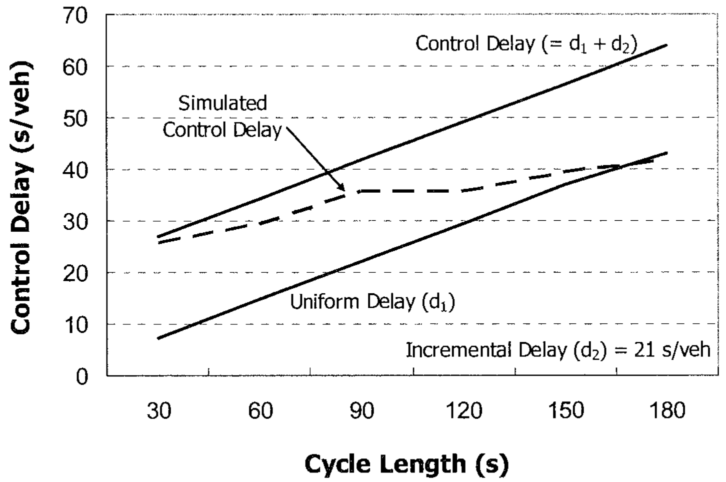
Conceptual Differences That Preclude Direct Comparison of Results

Conceptual differences in modeling approach may preclude the direct comparison of performance measures from the automobile methodology with those from alternative tools. The treatment of random arrivals is a case in point. There is a common misconception among analysts that alternative tools treat random arrivals in a similar manner.

A simple case is used to demonstrate the different ways alternative tools model random arrivals. Consider an isolated intersection with a two-phase sequence. The subject intersection approach serves only a through movement; there are no turning movements from upstream intersections or driveways. The only parameter that is allowed to vary in this example is the cycle length (all other variables are held constant).

The results of this experiment are shown in Exhibit 18-35. The two solid lines represent delay estimates obtained from the automobile methodology. Uniform delay is shown to increase linearly with cycle length. Incremental delay is constant with respect to cycle length because the volume-to-capacity ratio is constant. As a result, control delay (being the sum of the uniform and incremental delay) is also shown to increase linearly with cycle length.

Exhibit 18-35
Effect of Cycle Length on Delay



The dashed line represents the control delay estimate obtained from a simulation-based analysis tool. The simulation-based tool shows close agreement with the automobile methodology for short cycles but deviates for longer cycles. There are likely to be explainable reasons for this difference; however, the point is that such differences are likely to exist among tools. The analyst should understand the underlying modeling assumptions and limitations inherent in any tool (including the automobile methodology) when it is used. Moreover, the analyst should fully understand the definition of any performance measure used so as to interpret the results and observed trends properly.

Adjustment of Alternative Tool Parameters

For applications in which either an alternative tool or the automobile methodology can be used, some adjustment is generally required for the alternative tool if some consistency with the automobile methodology is desired. For example, the parameters that determine the capacity of a signalized approach (e.g., saturation flow rate and start-up lost time) should be adjusted to ensure that the simulated lane group (or approach) capacities match those estimated by the automobile methodology.

Step-by-Step Recommendations for Applying Alternative Tools

This part provides recommendations specifically for signalized intersection evaluation. The following steps should be taken to apply an alternative tool for signalized intersection analysis:

1. Determine whether the automobile methodology can provide a realistic assessment of the capacity and control delay for the signalized approaches of interest. The limitations stated at the end of Section 1 provide a good starting point for this assessment. If there are no conditions outside these limitations, then it should not be necessary to consider alternative tools. Otherwise, proceed with the remaining steps.

2. Select the appropriate tool in accordance with the general guidelines presented in Chapter 7.
3. Enter all available input characteristics and parameters.
4. Use the tool to evaluate the intersection. Be careful to observe the guidance provided in Chapter 7 regarding self-aggravating conditions that occur near capacity. If the tool is simulation based, then estimate the required number of runs so that the comparison is statistically valid.
5. If the documented delay definition and computational methodology used by the tool conform to the specifications set forth in Chapter 7 of this manual, then the delay estimates should be suitable for estimating the LOS. Otherwise, no such estimate should be attempted.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 31 includes example problems that address the following conditions:

- Left-turn storage bay overflow,
- RTOR operation,
- Short through lanes, and
- Closely spaced intersections.

4. EXAMPLE PROBLEMS

INTRODUCTION

This part of the chapter describes the application of each of the automobile, pedestrian, and bicycle methodologies through the use of example problems. Exhibit 18-36 provides an overview of these problems. The examples focus on the operational analysis level. The planning and preliminary engineering analysis level is identical to the operational analysis level in terms of the calculations, except that default values are used when field-measured values are not available.

Exhibit 18-36 Example Problems

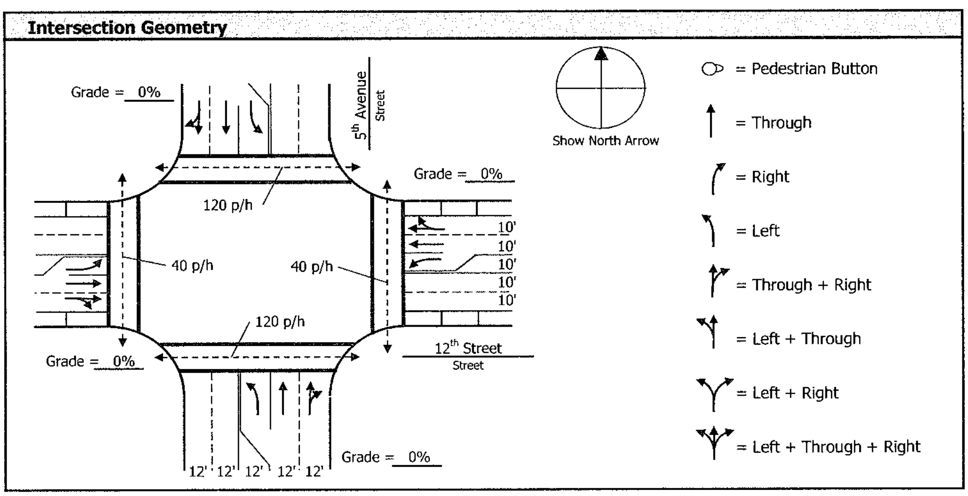
Problem Number	Description	Analysis Level
1	Automobile LOS	Operational
2	Pedestrian LOS	Operational
3	Bicycle LOS	Operational

EXAMPLE PROBLEM 1: AUTOMOBILE LOS

The Intersection

The intersection of 5th Avenue and 12th Street is an intersection of two urban arterial streets. It is shown in Exhibit 18-37.

Exhibit 18-37 Example Problem 1: Intersection Plan View



The Question

What is the motorist delay and LOS during the analysis period for each lane group and the intersection as a whole?

The Facts

The intersection's traffic, geometric, and signalization conditions are listed in Exhibit 18-38 and Exhibit 18-39.

Exhibit 18-38
Example Problem 1: Signal Conditions

Controller Data Worksheet			
General Information			
Analyst:	BR	Intersection:	5th Avenue/12th Street
Agency or Company:		Area Type: CBD	Phase 2: EB
Date Performed:	2/11/2010		
Analysis Time Period:	5:30 pm to 5:45 pm	Analysis Year:	2010
Filename: C:\Documents and Settings\TexasEX3			
Phase Sequence and Left-Turn Mode			
WB left (1) with WB thru (6)	EB left (5) with EB thru (2)	NB left (3) before SB thru (4)	SB left (7) before NB thru (8)
WB left permitted	EB left permitted	NB left (3) prot-perm	SB left (7) prot-perm

Phase Settings

Approach	Eastbound		Westbound		Northbound		Southbound	
Phase number		2		6	3	8	7	4
Movement		L+T+R		L+T+R	L	T+R	L	T+R
Lead/lag left-turn phase		--		--	Lead	--	Lead	--
Left-turn mode		Perm.		Perm.	Pr/Pm	--	Pr/Pm	--
Passage time, s		2.0		2.0	2.0	2.0	2.0	2.0
Maximum green, s		30		30	25	50	25	50
Minimum green, s		5		5	5	5	5	5
Yellow change, s		4.0		4.0	4.0	4.0	4.0	4.0
Red clearance, s		0.0		0.0	0.0	0.0	0.0	0.0
Walk+ ped. clear, s		19		19		21		21
Recall?:	No	No	No	No	No	No	No	No
Dual entry	No	Yes	No	Yes	No	Yes	No	Yes

Enable Simultaneous Gap-Out (check = Yes)? Phase Group 1,2,5,6: ☑ Phase Group 3,4,7,8: ☑				Enable Dallas Left-Turn Phasing? Phases 1,2,5,6: ☐ Phases 3,4,7,8: ☐				
Protected right-turn with left-turn phase?	n.a. / No	.	n.a. / No	.	Eastbd. right / No	.	Westbd. right / No	.
Phase number assignment to timers (by ring):				Controller timer ring structure:				
Ring 1:	0	2	3	4	Ring 1:	Timer 1	Timer 2	Timer 3	Timer 4
Ring 2:	0	6	7	8	Ring 2:	Timer 5	Timer 6	Timer 7	Timer 8

Exhibit 18-39
Example Problem 1: Traffic and Geometric Conditions

Movement-Specific Intersection Data Worksheet

Approach	Eastbound			Westbound			Northbound			Southbound		
Movement	L	T	R	L	T	R	L	T	R	L	T	R
Movement number	5	2	12	1	6	16	3	8	18	7	4	14
Traffic Characteristics *(Enter the volume data in all columns. For all other blue cells, enter values only if there are one or more lanes.)*												
Volume, veh/h	71	318	106	118	600	24	133	1,644	111	194	933	111
Right-turn-on-red volume, veh/h			0			0			22			33
Percent heavy vehicles, %	5	5		5	5		2	2		2	2	
Lane utilization adjustment factor	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Peak hour factor 1.00	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Start-up lost time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Extension of eff. green time, s	2.0	2.0		2.0	2.0		2.0	2.0		2.0	2.0	
Platoon ratio	1.000	1.000		1.000	1.000		1.000	1.000		1.000	1.000	
Upstream filtering factor	1.00	1.00		1.00	1.00		1.00	1.00		1.00	1.00	
Pedestrian volume, p/h		120			120			40			40	
Bicycle volume, bicycles/h		0			0			0			0	
(future use)												
Initial queue, veh	0	0		0	0		0	0		0	0	
Speed limit, mph	35	35	35	35	35	35	35	35	35	35	35	35
(future use)												
Multiple-Period Analysis Counts *(if all cell values = 0, then values in the 'Volume' row above will be used for a single-period analysis)*												
Period **1** traffic count, veh												
Period **2** traffic count, veh												
Period **3** traffic count, veh												
Period **4** traffic count, veh												
Intersection Approach Characteristics *(Enter the number of lanes. For all blue cells, enter values only if there are one or more lanes.)*												
Number of lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Average lane width, ft	10.0	10.0		10.0	10.0		12.0	12.0		12.0	12.0	
Number of receiving lanes		2			2			2			2	
Turn bay or segment length, ft	200	999		200	999		200	999		200	999	
Approach Data	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*	*Left Side*		*Right Side*
Parking present?	No		Yes	No		Yes	No		No	No		No
Parking maneuvers, maneuvers/h	0		5	0		5	0		0	0		0
Bus stopping rate, buses/h			0			0			0			0
Approach grade, %	0	0	0	0	0	0	0	0	0	0	0	0
Detection Data *(Enter values only if there are one or more lanes.)*												
Stop line detector length, ft	40	40	n.a.	40	40	n.a.	40	40	n.a.	40	40	n.a.
(future use)												

The intersection is located in a central business district–type environment. Adjacent signals are somewhat distant so the intersection is operated by using fully actuated control. Vehicle arrivals to each approach are characterized as "random" and are described by using a platoon ratio of 1.0.

The left-turn movements on the north–south street operate under protected-permitted control and lead the opposing through movements (i.e., a lead–lead phase sequence). The left-turn movements on the east–west street operate as permitted.

All intersection approaches have a 200-ft left-turn bay, an exclusive through lane, and a shared through and right-turn lane. The average width of the traffic lanes on the east–west street is 10 ft. The average width of the traffic lanes on the north–south street is 12 ft.

Crosswalks are provided on each intersection leg. A two-way flow rate of 120 p/h is estimated to use each of the east–west crosswalks and a two-way flow rate of 40 p/h is estimated to use each of the north–south crosswalks.

On-street parking is present on the east–west street. It is estimated that parking maneuvers on each intersection approach occur at a rate of 5 maneuvers/h during the analysis period.

The speed limit is 35 mi/h on each intersection approach. The analysis period is 0.25 h. There is no initial queue for any movement.

As noted in the next section, none of the lane groups at the intersection has two or more exclusive lanes. For this reason, the saturation flow rate adjustment factor for lane utilization is equal to 1.0 for all approaches. Any unequal lane use that may occur due to the shared through and right-turn lane groups will be accounted for in the lane group flow rate calculation, as described in Chapter 31.

Outline of Solution

Movement-Based Data

Exhibit 18-40 provides a summary of the analysis of the individual traffic movements at the intersection. The movement numbers shown follow the numbering convention in Exhibit 18-2.

Exhibit 18-40
Example Problem 1: Movement-Based Output Data

Movement:	EB L 5	EB T 2	EB R 12	WB L 1	WB T 6	WB R 16	NB L 3	NB T 8	NB R 18	SB L 7	SB T 4	SB R 14
Volume, veh/h	71	318	106	118	600	24	133	1,644	89	194	933	78
Initial Queue, veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj. Factor (A_pbT)	0.999		0.878	0.976		0.878	0.999		0.976	1.000		0.977
Parking, Bus Adj. Factors (f_bb x f_p)	1.000	1.000	0.875	1.000	1.000	0.875	1.000	1.000	1.000	1.000	1.000	1.000
Adjusted Sat. Flow Rate, veh/h/ln	1,629	1,629	1,629	1,629	1,629	1,629	1,676	1,676	1,676	1,676	1,676	1,676
Lanes	1	2	0	1	2	0	1	2	0	1	2	0
Lane Assignment	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.	L	TR	n.a.
Capacity, veh/h	147.23	629.27	201.44	205.81	853.60	34.08	326.46	1,545.51	83.10	224.96	1,604.59	134.14
Proportion Arriving On Green	0.294	0.294	0.294	0.294	0.294	0.294	0.061	0.491	0.491	0.097	0.527	0.527
Approach Volume, veh/h		495			742			1,866			1,205	
Approach Delay, s/veh		32.553			37.432			71.532			19.828	

Note: n.a. = not applicable

Two saturation flow rate adjustment factors are shown in Exhibit 18-40. One factor is the pedestrian–bicycle adjustment factor. This factor is used to estimate the saturation flow rate for the turn movement in a lane group. The "parking, bus adjustment factor" represents the product of the parking adjustment factor and the bus blockage adjustment factor. This combined factor is computed separately for the lane group that is adjacent to the parking or bus stop.

The adjusted saturation flow rate represents the saturation flow rate for all lane groups on the approach. It reflects the combined effect of lane width, heavy-vehicle presence, grade, and area type. The effect of pedestrians, bicycles, parking, bus blockage, lane utilization, right-turn maneuvers, and left-turn maneuvers is calculated separately at a later stage of the analysis because their values are influenced by signal timing, lane group demand flow rate, and lane group location (adjacent to parking or not, etc.). As such, these factors are internal to the iterative sequence of calculations used to estimate signal phase duration.

Capacity for a movement is computed by using the movement volume proportion in the lane group, lane group saturation flow rate, and corresponding phase duration. This variable represents the capacity of the movement, regardless of whether it is served in an exclusive lane or in a shared lane. If the movement is served in a shared lane, then the movement capacity represents the portion of the lane group capacity available to the movement, as distributed in proportion to the flow rate of the movements served by the associated lane group.

The last two rows in Exhibit 18-40 represent summary statistics for the approach. The approach volume represents the sum of the three movement volumes. Approach delay is computed as volume-weighted average for the lane groups served on an intersection approach.

Timer-Based Phase Data

Exhibit 18-41 provides a summary of the output data by using a signal controller perspective. The controller has eight timing functions (or timers), with Timers 1 to 4 representing Ring 1 and Timers 5 to 8 representing Ring 2. The ring structure and phase assignments were previously shown at the bottom of Exhibit 18-38. Timers 1 and 5 are not used at this intersection.

Exhibit 18-41
Example Problem 1: Timer-Based Phase Output Data

Timer Data								
Timer:	**1**	**2** **EB** **L.T.T+R**	**3** **NB** **L**	**4** **SB** **T.T+R**	**5**	**6** **WB** **L.T.T+R**	**7** **SB** **L**	**8** **NB** **T.T+R**
Assigned Phase		2	3	4		6	7	8
Case No		6	1	4		6	1	4
Phase Duration (G+Y+Rc), s		34.00	10.21	57.66		34.00	13.87	54.00
Change Period (Y+Rc), s		4.00	4.00	4.00		4.00	4.00	4.00
Max. Allowable Headway (MAH), s		3.44	3.13	3.06		3.44	3.13	3.06
Maximum Green Setting (Gmax), s		30.00	25.00	50.00		30.00	25.00	50.00
Max. Queue Clearance Time (g_c+l1), :		31.10	6.16	23.29		29.51	9.61	52.00
Green Extension Time (g_e), s		0.000	0.199	7.831		0.238	0.296	0.000
Probability of Phase Call (p_c)		1.000	0.977	1.000		1.000	0.996	1.000
Probability of Max Out (p_x)		1.000	0.000	0.179		1.000	0.000	1.000
Equilibrium Cycle Length, s: 102								

The timing function construct is essential in modeling a ring-based signal controller. *Timers* always occur in the same numeric sequence (i.e., 1 then 2 then 3 then 4 in Ring 1; 5 then 6 then 7 then 8 in Ring 2). The practice of associating movements to phases (e.g., the major-street through movement to Phase 2) coupled with the occasional need for lagging left-turn phases and split phasing creates the situation in which *phases* do not always time in sequence. For example, with a lagging left-turn phase sequence, major-street through Phase 2 times first and then major-street left-turn Phase 1 times second.

The modern controller accommodates the assignment of phases to timing functions by allowing the ring structure to be redefined manually or by time-of-day settings. Specification of this structure is automated in the computational engine by assigning phases to timers.

The methodology is based on modeling *timers*, not by directly modeling movements or phases. The methodology converts movement and phase input data into timer input data. It then models controller response to these inputs and computes timer duration and related performance measures.

The signalized intersection in this example problem has a lead–lead left-turn phase sequence on the north–south street. Hence, the timer numbers for this street are the same as the phase numbers, which are the same as the movement numbers (e.g., the northbound left-turn Movement 3 is associated with Phase 3, which is assigned to Timer 3). In contrast, the east–west street does not have left-turn phases, so one timer and one phase are used to serve all movements on a given approach.

The case number shown in Exhibit 18-41 is used as a single variable descriptor of each possible combination of left-turn mode and lane-group type (i.e., shared or exclusive). An understanding of this variable is not needed to interpret the output data.

The phase duration shown in the exhibit represents the estimated average phase duration during the analysis period. It represents the sum of the green, yellow change, and red clearance intervals. For Timer 2 (i.e., Phase 2), the average green interval duration is 30 s (= 34.00 – 4.00).

The durations of Phases 2, 3, and 4 add to the average cycle length of 101.87 s (= 34.00 + 10.21 + 57.66). Similarly, the durations of Phases 6, 7, and 8 add to the cycle length.

The cycle length is described in Exhibit 18-41 to be the "equilibrium" cycle length. The equilibrium cycle length is the average cycle length when all phase durations are dictated by traffic demand. However, the duration of several phases at this intersection is constrained by their maximum green limit. As such, the cycle length shown is not truly an equilibrium cycle length for this particular intersection.

The maximum green setting is input by the analyst. If the intersection were operated as coordinated-actuated, the "equivalent" maximum green setting would be shown here. It would be computed from the input phase splits and would reflect the specified force mode.

The maximum queue clearance time represents the largest queue clearance time of all lane groups served by the phase. Queue clearance time represents the time between the start of the green interval and the end of the queue service period. It is determined from the queue accumulation polygon. It includes the start-up lost time.

The maximum allowable headway, maximum green, and maximum queue clearance time apply only to actuated phases. They are not relevant to calculation of coordinated phase duration.

The green extension time represents the time the green interval is extended by arriving vehicles. This value is 0.0 s for two timers because they terminate by extension to their maximum limit (i.e., max-out).

The probability of a phase call represents the probability that one or more vehicles will place a call for service on the associated timer. The probability of max-out represents the probability that the phase will extend to the maximum green setting and terminate, perhaps leaving some unserved vehicles on the intersection approach.

Timer-Based Movement Data

Exhibit 18-42 summarizes the output for the vehicle movements assigned to each timer. Separate sections of output are shown in the exhibit for the left-turn, through, and right-turn movements. The assigned movement row identifies the movement (previously identified in Exhibit 18-40) assigned to each timer.

Exhibit 18-42
Example Problem 1: Timer-Based Movement Output Data

Timer Data								
Timer:	**1**	**2** **EB** **L.T.T+R**	**3** **NB** **L**	**4** **SB** **T.T+R**	**5**	**6** **WB** **L.T.T+R**	**7** **SB** **L**	**8** **NB** **T.T+R**
Left-Turn Movement Data								
Assigned Movement		5	3			1	7	
Mvmt. Sat Flow, veh/h		696.73	1,592.65			818.40	1,592.65	
Through Movement Data								
Assigned Movement		2		4		6		8
Mvmt. Sat Flow, veh/h		2,136.77		3,046.34		2,898.49		3,148.76
Right-Turn Movement Data								
Assigned Movement		12		14		16		18
Mvmt. Sat Flow, veh/h		684.02		254.67		115.71		169.31

The saturation flow rate shown in Exhibit 18-42 represents the saturation flow rate computed for the movement. For through movements in exclusive lanes, the movement saturation flow rate is equal to the number of through lanes times the adjusted saturation flow rate, times the pedestrian–bicycle adjustment factor, times the combined parking–bus blockage adjustment factor. For turn movements in exclusive lanes, the calculation is similar except that the left-turn (or right-turn) adjustment factor is also applied.

For turn movements that share a lane with a through movement, the saturation flow rate for the lane group is computed by using the procedure described in Chapter 31. The movement saturation flow rate represents the portion of the lane group saturation flow rate available to the movement, as distributed in proportion to the flow rate of the movements served by the lane group. To illustrate this point, consider Timer 4. It has a shared-lane lane group with 15.7% right-turning vehicles, 84.3% through vehicles, and a saturation flow rate of 1,624.5 veh/h/ln. The turn movement saturation flow rate is 254.67 veh/h (= 0.157 × 1,624.5). The through movement saturation flow rate in this shared lane is 1,369.8 veh/h (= 0.843 × 1,624.5). The through movement is also served by one exclusive through lane with a saturation flow rate of 1,676.5 veh/h. Thus, the total through-movement saturation flow rate is 3,046.3 veh/h (= 1,369.8 + 1,676.5). The individual lane group saturation flow rates used in this example were obtained from the lane group data described in the next few sections.

Timer-Based Left Lane Group Data

Exhibit 18-43 summarizes the output for the "left" lane group associated with an intersection approach. Each left lane group includes the left-turn movements when they exist on an intersection approach. A left lane group will also contain all the output data for a single-lane approach, regardless of whether a left-turn movement exists.

The "lane assignment" row indicates the lane groups served by the timer (e.g., L, left turn; T, through; R, right turn). The letter "L" is shown for Timers 2 and 6 as a reminder that the timer is serving a left-turn lane group. Other letter combinations are possible. For example, "L+T" indicates the timer is serving a lane group consisting of a shared lane serving left-turn and through movements. A "L+T+R" sequence indicates a single-lane approach serving all movements.

Exhibit 18-43
Example Problem 1: Timer-Based Left Lane Group Output Data

Timer Data								
Timer:	**1**	**2 EB L.T+R**	**3 NB L**	**4 SB T.T+R**	**5**	**6 WB L.T+R**	**7 SB L**	**8 NB T.T+R**
Left Lane Group Data								
Assigned Movement		5	3			1	7	
Lane Assignment		L	L (Pr/Pm)			L	L (Pr/Pm)	
Lanes in Group		1	1			1	1	
Group Volume (v), veh/h		71.0	133.0			118.0	194.0	
Group Sat. Flow (s), veh/h/ln		696.7	1,592.6			818.4	1,592.6	
Queue Serve Time (g_s), s		10.289	4.160			14.328	7.613	
Cycle Queue Clear Time (g_c), s		29.097	4.160			27.508	7.613	
*Perm LT Sat Flow Rate (s_l), veh/h/l		696.7	499.3			818.4	250.4	
*Shared LT Sat Flow (s_sh), veh/h/ln		0.0	0.0			0.0	0.0	
*Perm LT Eff. Green (g_p), s		30.00	50.00			30.00	55.31	
*Perm LT Serve Time (g_u), s		11.19	32.37			16.82	0.00	
*Perm LT Que Serve Time (g_ps), s		10.29	6.40			14.33	0.00	
*Time to First Blk (g_f), s		0.00	0.00			0.00	0.00	
*Serve Time pre Blk (g_fs), s		0.00	0.00			0.00	0.00	
*Proportion LT Inside Lane (P_L)		1.000	1.000			1.000	1.000	
Lane Group Capacity (c), veh/h		147.2	326.5			205.8	225.0	
Volume-to-Capacity Ratio (X)		0.482	0.407			0.573	0.862	
Available Capacity (c_a), veh/h		147.2	620.2			205.8	461.5	
Upstream Filter Factor (I)		1.000	1.000			1.000	1.000	
Uniform Delay (d1), s/veh		44.936	13.243			41.483	30.229	
Incremental Delay (d2), s/veh		0.910	0.304			2.496	3.791	
Initial Queue Delay (d3), s/veh		0.000	0.000			0.000	0.000	
Control Delay (d), s/veh		45.846	13.547			43.979	34.020	
First-Term Queue (Q1), veh/ln		1.75	1.39			2.83	2.98	
Second-Term Queue (Q2), veh/ln		0.04	0.03			0.14	0.24	
Third-Term Queue (Q3), veh/ln		0.00	0.00			0.00	0.00	
Percentile bk-of-que factor (f_B%)		1.00	1.00			1.00	1.00	
Percentile Back of Queue (Q%), veh/ln		1.78	1.42			2.97	3.22	
Percentile Storage Ratio (RQ%)		0.232	0.180			0.386	0.409	
Initial Queue (Qb), veh		0.0	0.0			0.0	0.0	
Final (Residual) Queue (Qe), veh		0.0	0.0			0.0	0.0	
Saturated Delay (ds), s/veh		0.000	0.000			0.000	0.000	
Saturated Queue (Qs), veh		0.00	0.00			0.00	0.00	
Saturated Capacity (cs), veh/h		0.0	0.0			0.0	0.0	
Initial Queue Clear Time (tc), h		0.000	0.000			0.000	0.000	

The lane assignment row also indicates the operational mode for the left-turn movements. "Prot" indicates a protected left-turn mode. "Pr/Pm" indicates a protected-permitted left-turn mode. Other designations with the letter "L" indicate either a permitted left-turn mode or split phasing.

The rows listed in Exhibit 18-43 that start with "queue serve time" and end with "uniform delay" correspond to variables that are computed from the queue accumulation polygon.

The permitted left-turn saturation flow rate represents the filtering flow rate of a permitted left-turn movement. Equations for computing this flow rate and the other variables identified with an asterisk (*) are described in Chapter 31.

The shared left-turn saturation flow rate is the saturation flow rate of a shared left-turn and through lane during the period after the first blocking left-turning vehicle arrives but before the queue service ends. This flow rate is applicable only when the opposing approach has one traffic lane. It reflects the opportunities to serve the subject approach that are created by left-turning vehicles in the opposing lane.

The permitted left-turn effective green time represents the time available for permitted left-turn movement. In general, it is the time in the opposing through movement phase that is associated with a permissive green ball signal indication. Its duration can vary with phase sequence and timing.

The permitted left-turn service time represents the time required to serve the left-turn queue. This time occurs during the permitted left-turn effective green time but after the conflicting queue clears. It exists for phases that operate in the permitted mode or in the protected-permitted mode.

The time to first block applies to a lane group with a shared lane and a left-turn movement that operates in the permitted or protected-permitted mode. It represents the time from the start of the through phase until the first left-turning vehicle arrives at the stop line and stops to wait for an acceptable gap in oncoming traffic.

The queue service time before the first block (i.e., serve time pre blk) represents the queue service time for a stream of through movements in a shared left-turn and through lane. If the left-turn flow rate is low, the time to first block may occur well into the phase. In this case, it is possible that the queue of through vehicles in the shared lane will be served before the first left-turning vehicle arrives. This variable applies only to lane groups with a shared left-turn lane.

The proportion of left-turning vehicles in the inside lane represents the distribution of vehicles in the left-lane group. If a left-turn bay exists, then the proportion equals 1.0. If the lane group is shared by left-turn and through movements, then the proportion can vary between 0.0 and 1.0. If it is 1.0, then the shared lane operates as an exclusive left-turn lane.

Uniform delay represents the area under the queue accumulation polygon. This polygon is based on an average arrival rate during the green indication and an average arrival rate during the red indication. As such, it reflects the effect of progression on the delay estimate.

The available capacity is computed for all actuated phases and noncoordinated phases. It is computed by using the maximum green setting for the phase. For coordinated phases, the available capacity is computed by using the average effective green time.

The incremental delay is computed by using the incremental delay equation. For actuated phases, it uses available capacity to estimate the incremental delay factor *k*. For coordinated phases and phases set to "recall-to-maximum," it uses a factor of 0.50.

The first-term queue is a back-of-queue estimate that is obtained from an arrival–departure polygon. This polygon is based on the specification of arrival rates during the red and green intervals. As such, it reflects the effect of progression on first-term queue size. The procedure for developing this polygon is described in Chapter 31.

The second-term queue is computed as a derivative of the incremental delay estimate. It represents the average number of vehicles in queue each cycle due to random variation in arrivals plus those vehicles in queue due to oversaturation during the analysis period.

The queue storage ratio represents the ratio of the back-of-queue size to the available storage length. In general, this ratio can be computed for turn bays and through lanes; however, it is computed only for the left-turn bays in this example. A value of 0.0 indicates that no turn vehicles are queued in the bay. A value of 1.0 or more indicates that the queue completely fills the bay at some point during the cycle.

The initial queue reflects the input initial queue value when a single analysis period is evaluated. In contrast, it reflects the residual queue from the previous analysis period for the second and subsequent analysis periods of a multiple-period analysis.

The saturated delay, queue, and capacity data reflect the output from a complete (and separately computed) intersection analysis. For this separate analysis, lane groups with an initial queue will have their demand flow rate adjusted so that volume equals lane group capacity. The saturated delay equals the uniform delay computed for this "saturated" condition. Similarly, the "saturated" queue equals the first-term queue for the saturated condition.

The initial queue clear time indicates the time when the last vehicle that arrives at an overflow queue during the analysis period clears the intersection (measured from the start of the analysis period).

Timer-Based Middle Lane Group Data

Exhibit 18-44 provides a summary of the output for the "middle" lane group associated with an intersection approach. This lane group is used when one or more exclusive lanes serve through vehicles on an intersection approach. The explanation of the various output statistics is the same as that previously given for the left lane groups.

In Exhibit 18-44, the exclusive through lane served by Timer 8 has a volume-to-capacity ratio that slightly exceeds 1.0. This condition results in a large value of control delay (= 73.6 s/veh) and a final (i.e., residual) queue size of 11.8 veh. The last vehicle to arrive at this queue during the analysis period will depart the intersection 0.264 h after the *start* of the 0.25-h analysis period.

Timer-Based Right Lane Group Data

Exhibit 18-45 summarizes the output for the "right" lane group associated with an intersection approach. This lane group is used when there are two or more lanes on an intersection approach and a through or right-turn movement is present. A lane that is shared by the right-turn and through movements is always shown in the right lane group. The explanation of the various output statistics is the same as that previously given for the left lane groups.

The protected right-turn saturation flow rate row is used when the right-turn movement is provided a green arrow indication concurrently with its complementary left-turn phase on the cross street. This flow rate represents the saturation flow rate during the green arrow. Similarly, the protected right-turn effective green time equals the effective green time coincident with the green arrow indication. This operation is not provided at the subject intersection, so the values for these two variables equal 0.0.

Exhibit 18-44
Example Problem 1: Timer-Based Middle Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Middle Lane Group Data								
Assigned Movement		2		4		6		8
Lane Assignment		T		T		T		T
Lanes in Group		1		1		1		1
Group Volume (v), veh/h		239.2		513.4		336.6		870.1
Group Sat. Flow (s), veh/h/ln		1,628.6		1,676.5		1,628.6		1,676.5
Queue Serve Time (g_s), s		12.376		21.284		18.724		50.000
Cycle Queue Clear Time (g_c), s		12.376		21.284		18.724		50.000
Lane Group Capacity (c), veh/h		479.6		883.0		479.6		822.9
Volume-to-Capacity Ratio (X)		0.499		0.581		0.702		1.057
Available Capacity (c_a), veh/h		479.6		883.0		479.6		822.9
Upstream Filter Factor (I)		1.000		1.000		1.000		1.000
Uniform Delay (d1), s/veh		29.717		16.445		31.956		25.934
Incremental Delay (d2), s/veh		0.299		0.649		3.876		47.658
Initial Queue Delay (d3), s/veh		0.000		0.000		0.000		0.000
Control Delay (d), s/veh		30.017		17.094		35.832		73.592
First-Term Queue (Q1), veh/ln		4.73		7.61		7.15		18.26
Second-Term Queue (Q2), veh/ln		0.04		0.16		0.52		10.89
Third-Term Queue (Q3), veh/ln		0.00		0.00		0.00		0.00
Percentile bk-of-que factor (f_B%)		1.00		1.00		1.00		1.00
Percentile Back of Queue (Q%), veh/ln		4.77		7.77		7.67		29.15
Percentile Storage Ratio (RQ%)		0.124		0.198		0.200		0.741
Initial Queue (Qb), veh		0.0		0.0		0.0		0.0
Final (Residual) Queue (Qe), veh		0.0		0.0		0.0		11.8
Saturated Delay (ds), s/veh		0.000		0.000		0.000		0.000
Saturated Queue (Qs), veh		0.00		0.00		0.00		0.00
Saturated Capacity (cs), veh/h		0.0		0.0		0.0		0.0
Initial Queue Clear Time (tc), h		0.000		0.000		0.000		0.264

Exhibit 18-45
Example Problem 1: Timer-Based Right Lane Group Output Data

Timer Data								
Timer:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		EB	**NB**	**SB**		**WB**	**SB**	**NB**
		L.T.T+R	**L**	**T.T+R**		**L.T.T+R**	**L**	**T.T+R**
Right Lane Group Data								
Assigned Movement		12		14		16		18
Lane Assignment		T+R		T+R		T+R		T+R
Lanes in Group		1		1		1		1
Group Volume (v), veh/h		184.8		497.6		287.4		862.9
Group Sat. Flow (s), veh/h/ln		1,192.2		1,624.5		1,385.6		1,641.6
Queue Serve Time (g_s), s		13.179		21.285		18.808		50.000
Cycle Queue Clear Time (g_c), s		13.179		21.285		18.808		50.000
*Prot RT Sat Flow Rate (s_R), veh/h/ln		0.000		0.000		0.000		0.000
*Prot RT Eff. Green (g_R), s		0.000		0.000		0.000		0.000
*Proportion RT Outside Lane (P_R)		0.574		0.157		0.084		0.103
Lane Group Capacity (c), veh/h		351.1		855.7		408.1		805.7
Volume-to-Capacity Ratio (X)		0.526		0.581		0.704		1.071
Available Capacity (c_a), veh/h		351.1		855.7		408.1		805.7
Upstream Filter Factor (I)		1.000		1.000		1.000		1.000
Uniform Delay (d1), s/veh		30.001		16.445		31.986		25.934
Incremental Delay (d2), s/veh		0.729		0.670		4.631		52.458
Initial Queue Delay (d3), s/veh		0.000		0.000		0.000		0.000
Control Delay (d), s/veh		30.729		17.116		36.617		78.392
First-Term Queue (Q1), veh/ln		3.68		7.37		6.11		17.88
Second-Term Queue (Q2), veh/ln		0.07		0.16		0.52		11.74
Third-Term Queue (Q3), veh/ln		0.00		0.00		0.00		0.00
Percentile bk-of-que factor (f_B%)		1.00		1.00		1.00		1.00
Percentile Back of Queue (Q%), veh/ln		3.76		7.53		6.64		29.62
Percentile Storage Ratio (RQ%)		0.098		0.192		0.173		0.753
Initial Queue (Qb), veh		0.0		0.0		0.0		0.0
Final (Residual) Queue (Qe), veh		0.0		0.0		0.0		14.3
Saturated Delay (ds), s/veh		0.000		0.000		0.000		0.000
Saturated Queue (Qs), veh		0.00		0.00		0.00		0.00
Saturated Capacity (cs), veh/h		0.0		0.0		0.0		0.0
Initial Queue Clear Time (tc), h		0.000		0.000		0.000		0.268

Results

A comparison of the lane-group volumes in Exhibit 18-43, Exhibit 18-44, and Exhibit 18-45 indicates the extent to which drivers are expected to distribute themselves among the lane groups on each intersection approach. For example, Timer 2 serves three lane groups on the eastbound approach. The left lane group is an exclusive lane and serves all left-turn movements. The middle lane group serves 239 veh/h of the 318 veh/h in the through movement (i.e., about 75%). The right lane group serves the remaining through vehicles (i.e., 79 veh/h) and the right-turning vehicles (106 veh/h) for a total flow rate of 185 veh/h. There are fewer vehicles in the right lane group (i.e., 185 versus 239) because some through drivers choose the middle lane to avoid any possible delay that might be incurred by the presence of right-turning vehicles in the outside lane.

Exhibit 18-46 summarizes the delay for each lane group, approach, and the intersection as a whole. It also provides the volume-to-capacity ratio and LOS for each lane group. The delay varies widely among lane groups, as does the LOS. The northbound through and right-turn movements have the highest delay and a LOS F condition.

Exhibit 18-46
Example Problem 1: Performance Measure Summary

Group:	EB Left L	EB Middle T	EB Right T+R	WB Left L	WB Middle T	WB Right T+R	NB Left L (Pr/Pm)	NB Middle T	NB Right T+R	SB Left L (Pr/Pm)	SB Middle T	SB Right T+R
Lane Group Summary												
Group Volume (v), veh/h	71.0	239.2	184.8	118.0	336.6	287.4	133.0	870.1	862.9	194.0	513.4	497.6
Volume-to-Capacity Ratio (X)	0.482	0.499	0.526	0.573	0.702	0.704	0.407	1.057	1.071	0.862	0.581	0.581
Control Delay (d), s/veh	45.846	30.017	30.729	43.979	35.832	36.617	13.547	73.592	78.392	34.020	17.094	17.116
Level of Service	D	C	C	D	D	D	B	F	F	C	B	B
Approach Summary												
Approach Volume, veh/h		495.0			742.0			1866.0			1205.0	
Approach Delay, s/veh		32.553			37.432			71.532			19.828	
Level of Service		C			D			E			B	
Intersection Summary												
Entering Volume, veh/h	4308.0											
Control Delay, s/veh	46.717											
Level of Service	D											

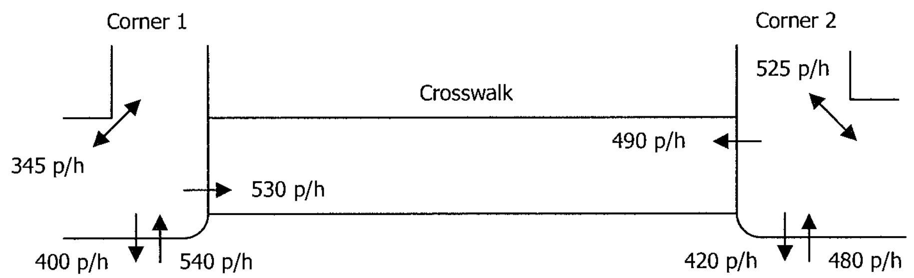
The fact that several phases are terminating by max-out and that the northbound through and right-turn movements are congested (i.e., Timer 8) suggests that some improvements could be made at this intersection. Simply increasing the maximum green settings is not a solution and, in fact, increases the overall delay and queue size for most lane groups. Physical changes to the intersection geometry to increase capacity could be considered.

EXAMPLE PROBLEM 2: PEDESTRIAN LOS

The Intersection

The pedestrian crossing of interest crosses the north leg at a signalized intersection. The north–south street is the minor street and the east–west street is the major street. The intersection serves all north–south traffic concurrently (i.e., no left-turn phases) and all east–west traffic concurrently. The signal has an 80-s cycle length. The crosswalk and intersection corners that are the subject of this example problem are shown in Exhibit 18-47.

Exhibit 18-47
Example Problem 2:
Pedestrian Flow Rates



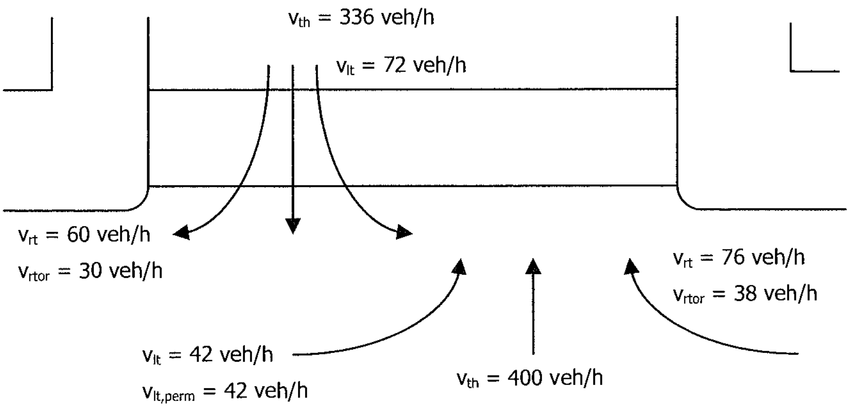
The Question

What is the pedestrian LOS for the crossing?

The Facts

Pedestrian flow rates are shown in Exhibit 18-47. Vehicular flow rates are shown in Exhibit 18-48.

Exhibit 18-48
Example Problem 2:
Vehicular Demand Flow Rates



In addition, the following facts are known about the crosswalk and the intersection corners:

Major street: Phase duration, $D_{p,mj} = 48$ s
Yellow change interval, $Y_{mj} = 4$ s
Red clearance interval, $R_{mj} = 1$ s
Walk setting, $Walk_{mj} = 7$ s
Pedestrian clear setting, $PC_{mj} = 8$ s
Four traffic lanes (no turn bays)

Minor street: Phase duration, $D_{p,mi} = 32$ s
Yellow change interval, $Y_{mi} = 4$ s
Red clearance interval, $R_{mi} = 1$ s
Walk setting, $Walk_{mi} = 7$ s
Pedestrian clear setting, $PC_{mi} = 13$ s
Two traffic lanes (no turn bays)
85th percentile speed at a midsegment location, $S_{85,mi} = 35$ mi/h

Corner 1: Total walkway width, $W_a = W_b = 16$ ft

Corner radius, $R = 15$ ft

Corner 2: Total walkway width, $W_a = W_b = 18$ ft

Corner radius, $R = 15$ ft

Other data: No right-turn channelizing islands provided on any corner

Effective crosswalk width, $W_c = 16$ ft

Crosswalk length, $L_c = 28$ ft

Walking speed, $S_p = 4$ ft/s

Pedestrian signal indications are provided for each crosswalk

Rest-in-walk mode is not used for any phase

Comments

On the basis of the variable notation in Exhibit 18-25, the subject crosswalk is "Crosswalk C" because it crosses the minor street. The outbound pedestrian flow rate v_{co} at Corner 1 equals inbound flow rate v_{ci} at Corner 2, and the inbound flow rate v_{ci} at Corner 1 equals the outbound flow rate v_{co} at Corner 2.

Outline of Solution

First, the circulation area is calculated for both corners. Next, the circulation area is calculated for the crosswalk. The street corner and crosswalk circulation areas are then compared with the qualitative descriptions of pedestrian space listed in Exhibit 18-24.

Pedestrian delay and the pedestrian LOS score are then calculated for the crossing. Finally, LOS for the crossing is determined on the basis of the computed score and the threshold values in Exhibit 18-5.

Computational Steps

Step 1: Determine Street Corner Circulation Area

A. Compute Available Time–Space

For Corner 1, the available time–space is computed with Equation 18-52.

$$TS_{\text{corner}} = C(W_a W_b - 0.215R^2)$$

$$TS_{\text{corner}} = (80)[(16)(16) - 0.215(15)^2]$$

$$TS_{\text{corner}} = 16{,}610 \text{ ft}^2\text{-s}$$

B. Compute Holding-Area Waiting Time

Because pedestrian signal indications are provided and rest-in-walk is not enabled, the effective walk time for the phase serving the major street is computed with Equation 18-49.

$$g_{\text{Walk},mj} = Walk_{mj} + 4.0$$

$$g_{\text{Walk},mj} = 7.0 + 4.0 = 11 \text{ s}$$

The number of pedestrians arriving at the corner during each cycle to cross the minor street is computed with Equation 18-54.

$$N_{co} = \frac{v_{co}}{3{,}600} C$$

$$N_{co} = \frac{530}{3{,}600}(80) = 11.8 \text{ p}$$

The total time spent by pedestrians waiting to cross the minor street during one cycle is then calculated with Equation 18-53.

$$Q_{tco} = \frac{N_{co}(C - g_{\text{Walk},mj})^2}{2C}$$

$$Q_{tco} = \frac{(11.8)(80 - 11)^2}{2(80)} = 350.5 \text{ p-s}$$

By the same procedure, the total time spent by pedestrians waiting to cross the major street during one cycle (Q_{tdo}) is found to be 264.5 p-s.

C. Compute Circulation Time–Space

The circulation time–space is found by using Equation 18-58.

$$TS_c = TS_{\text{corner}} - [5.0(Q_{tdo} + Q_{tco})]$$

$$TS_c = 16{,}610 - [5.0\ (350.5 + 264.5)] = 13{,}535 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Corner Circulation Area

The total number of circulating pedestrians is computed with Equation 18-60.

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600} C$$

$$N_{tot} = \frac{490 + 530 + 540 + 400 + 345}{3{,}600}(80) = 51.2 \text{ p}$$

Finally, the corner circulation area per pedestrian is calculated with Equation 18-59.

$$M_{\text{corner}} = \frac{TS_c}{4.0 N_{tot}}$$

$$M_{\text{corner}} = \frac{13{,}535}{4.0(51.2)} = 66.1 \text{ ft}^2/\text{p}$$

By following the same procedure, the corner circulation area per pedestrian for Corner 2 is found to be 87.6 ft^2/p. According to the qualitative descriptions provided in Exhibit 18-24, pedestrians at both corners will have the ability to move in the desired path, with no need to alter their movements to avoid conflicts.

Step 2: Determine Crosswalk Circulation Area

The analysis conducted in this step describes the circulation area for pedestrians in the subject crosswalk.

A. Establish Walking Speed

As given in the "facts" section, the average walking speed is determined to be 4.0 ft/s.

B. Compute Available Time–Space

Rest-in-walk is not enabled, so the pedestrian service time g_{ped} is estimated to equal the sum of the walk and pedestrian clear settings. The time–space available in the crosswalk is found with Equation 18-61.

$$TS_{cw} = L_c W_c \, g_{\text{Walk},mj}$$

$$TS_{cw} = (28)(16)(11) = 4{,}928 \text{ ft}^2\text{-s}$$

C. Compute Effective Available Time–Space

The number of turning vehicles during the walk and pedestrian clear intervals is calculated with Equation 18-64.

$$N_{tv} = \frac{v_{lt,perm} + v_{rt} - v_{rtor}}{3{,}600} C$$

$$N_{tv} = \frac{42 + 76 - 38}{3{,}600}(80) = 1.8 \text{ veh}$$

The time–space occupied by turning vehicles can then be computed with Equation 18-63.

$$TS_{tv} = 40 N_{tv} W_c$$

$$TS_{tv} = 40(1.8)(16) = 1{,}138 \text{ ft}^2\text{-s}$$

The effective available crosswalk time–space $TS_{cw}{}^*$ is found by subtracting the total available crosswalk time–space TS_{cw} from the time–space occupied by turning vehicles.

$$TS_{cw}{}^* = TS_{cw} - TS_{tv}$$

$$TS_{cw}{}^* = 4{,}928 - 1{,}138 = 3{,}790 \text{ ft}^2\text{-s}$$

D. Compute Pedestrian Service Time

The number of pedestrians exiting the curb when the WALK indication is presented is as follows:

$$N_{ped,co} = N_{co} \frac{C - g_{\text{Walk},mj}}{C}$$

$$N_{ped,co} = (11.8)\frac{80 - 11}{80} = 10.2 \text{ p}$$

Because the crosswalk width is greater than 10 ft, the pedestrian service time is computed as follows:

$$t_{ps,co} = 3.2 + \frac{L_c}{S_p} + 2.7\frac{N_{ped,co}}{W_c}$$

$$t_{ps,co} = 3.2 + \frac{28}{4.0} + 2.7\left(\frac{10.2}{16}\right) = 11.9 \text{ s}$$

The other travel direction in the crosswalk is analyzed next. The number of pedestrians arriving at Corner 1 each cycle by crossing the minor street is as follows:

$$N_{ci} = \frac{v_{ci}}{3{,}600} C$$

$$N_{ci} = \frac{490}{3{,}600}(80) = 10.9 \text{ p}$$

The sequence of calculations is repeated for this second travel direction in the subject crosswalk to indicate that $N_{ped,ci}$ is equal to 9.4 p and $t_{ps,ci}$ is 11.8.

E. Compute Crosswalk Occupancy Time

The crosswalk occupancy time for the crosswalk is computed as follows:

$$T_{occ} = t_{ps,co} N_{co} + t_{ps,ci} N_{ci}$$

$$T_{occ} = 11.9\ (11.8) + 11.8\ (10.9) = 268.6 \text{ p-s}$$

F. Compute Pedestrian Crosswalk Circulation Area

Finally, the crosswalk circulation area per pedestrian for the crosswalk is computed as follows:

$$M_{cw} = \frac{TS^*_{cw}}{T_{occ}}$$

$$M_{cw} = \frac{3{,}790}{268.6} = 14.1 \text{ ft}^2/\text{p}$$

The crosswalk circulation area is found to be 14.1 ft^2/p. According to the qualitative descriptions provided in Exhibit 18-24, pedestrians will find that their walking speed is restricted, with very limited ability to pass slower pedestrians. Improvements to the crosswalk should be considered and may include a wider crosswalk or a longer walk interval.

Step 3: Determine Pedestrian Delay

The pedestrian delay is calculated as follows:

$$d_p = \frac{(C - g_{\mathrm{Walk},mj})^2}{2C}$$

$$d_p = \frac{(80-11)^2}{2\,(80)} = 29.8 \text{ s/p}$$

Step 4: Determine Pedestrian LOS Score for Intersection

The number of vehicles traveling on the minor street during a 15-min period is computed as follows:

$$n_{15,mi} = \frac{0.25}{N_c} \sum v_i$$

$$n_{15,mi} = \frac{0.25}{2}(72+336+60+42+400+76) = 123.3 \text{ veh/ln}$$

The cross-section adjustment factor is calculated as follows:

$$F_w = 0.681(N_c)^{0.514}$$

$$F_w = 0.681(2)^{0.514} = 0.972$$

The motorized vehicle adjustment factor is computed as follows:

$$F_v = 0.00569\left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,c}(0.0027 n_{15,mi} - 0.1946)$$

$$F_v = 0.00569\left(\frac{30+42}{4}\right) - (0)\,[0.0027(123.3) - 0.1946] = 0.102$$

The motorized vehicle speed adjustment factor is then computed:

$$F_s = 0.00013\, n_{15,mi} S_{85,mi}$$

$$F_s = 0.00013(123.3)(35) = 0.561$$

The pedestrian delay adjustment factor is calculated as follows:

$$F_{\mathrm{delay}} = 0.0401 \ln(d_{p,c})$$

$$F_{\mathrm{delay}} = 0.0401 \ln(29.8) = 0.136$$

The pedestrian LOS score for the intersection $I_{p,int}$ is then computed as follows:

$$I_{p,int} = 0.5997 + F_w + F_v + F_S + F_{\mathrm{delay}}$$

$$I_{p,int} = 0.5997 + 0.972 + 0.102 + 0.561 + 0.136 = 2.37$$

For this crosswalk, $I_{p,int}$ is found to be 2.37.

Step 5: Determine LOS

According to Exhibit 18-5, the crosswalk operates at LOS B.

Discussion

The crosswalk was found to operate at LOS B in Step 5. It was determined in Step 1 that the pedestrians at both corners have adequate space to allow freedom of movement. Crosswalk circulation area was found to be restricted in Step 2 and improvements are probably justified. Moreover, the pedestrian delay computed in Step 3 was found to be slightly less than 30 s/p. With this much delay, some pedestrians may not comply with the signal indication.

EXAMPLE PROBLEM 3: BICYCLE LOS

The Intersection

A 5-ft-wide bicycle lane is provided at a signalized intersection.

The Question

What is the LOS of this bicycle lane?

The Facts

Saturation flow rate for bicycles = 2,000 bicycles/h

Effective green time = 48 s

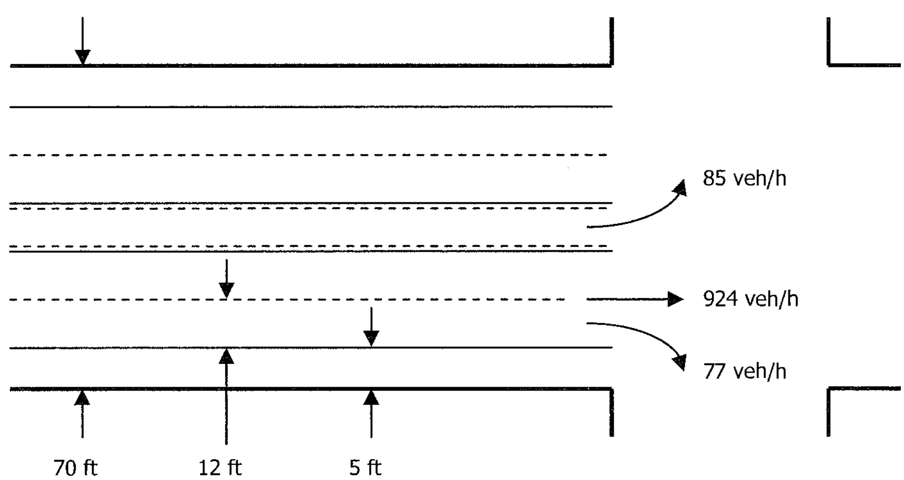
Cycle length = 120 s

Bicycle flow rate = 120 bicycles/h

No on-street parking

The vehicular flow rates and street cross-section element widths are as shown in Exhibit 18-49.

Exhibit 18-49
Example Problem 3: Vehicular Demand Flow Rates and Cross-Section Element Widths



Outline of Solution

Bicycle delay and the bicycle LOS score will be computed. LOS is then determined on the basis of the computed score and the threshold values in Exhibit 18-5.

Computational Steps

Step 1: Determine Bicycle Delay

A. Compute Bicycle Lane Capacity

The capacity of the bicycle lane is calculated with Equation 18-78:

$$c_b = s_b \frac{g_b}{C}$$

$$c_b = (2{,}000)\frac{48}{120} = 800 \text{ bicycles/h}$$

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 18-79:

$$d_b = \frac{0.5C(1 - g_b / C)^2}{1 - \frac{g_b}{C}\text{Min}\left[\frac{v_{bic}}{c_b}, 1.0\right]}$$

$$d_b = \frac{0.5(120)(1 - 48/120)^2}{1 - \frac{48}{120}\text{Min}\left[\frac{120}{800}, 1.0\right]} = 23.0 \text{ s/bicycle}$$

Step 2: Determine Bicycle LOS Score for Intersection

As shown in Exhibit 18-49, the total width of the outside through lane, bicycle lane, and paved shoulder is 17 ft (= 12 + 5 + 0). The cross-section adjustment factor can then be calculated with Equation 18-81:

$$F_w = 0.0153W_{cd} - 0.2144W_t$$

$$F_w = 0.0153(70) - 0.2144(17) = -2.57$$

The motor-vehicle volume adjustment factor must also be calculated, by using Equation 18-82:

$$F_v = 0.0066 \frac{v_{lt} + v_{th} + v_{rt}}{4N_{th}}$$

$$F_v = 0.0066 \frac{85 + 924 + 77}{4(2)} = 0.90$$

The bicycle LOS score can then be computed with Equation 18-80:

$$I_{b,int} = 4.1324 + F_w + F_v$$

$$I_{b,int} = 4.1324 - 2.57 + 0.90 = 2.45$$

Step 3: Determine LOS

According to Exhibit 18-5, this bicycle lane would operate at LOS B through the signalized intersection.

Discussion

The bicycle lane was found to operate at LOS B. The bicycle delay was found to be 23.0 s/bicycle, which is low enough that most bicyclists are not likely to be impatient. However, if the signal timing at the intersection were to be changed, the bicycle delay would need to be computed again to verify that it does not rise above 30 s/bicycle.

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CHAPTER 19
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, where one street—the *major street*—is uncontrolled, while the other street—the *minor street*—is controlled by STOP signs. The other typical configuration is a three-leg intersection, where 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. **Chapter 19, Two-Way STOP-Controlled Intersections,** 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 are considered a standard type of TWSC intersection, when the stem of the T is controlled by a STOP sign.

Capacity analysis of TWSC intersections requires a clear description and understanding of the interaction between travelers on the minor, or STOP-controlled, approach with travelers on the major street. Both gap acceptance and empirical 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*).

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 are located within 0.25 mi of a signalized intersection (for the major-street approaches). 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.

LEVEL-OF-SERVICE CRITERIA

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 (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 major-street left turns by using criteria given in Exhibit 19-1. 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 important LOS deficiencies for minor movements. As Exhibit 19-1 notes, LOS F is assigned to the movement if the volume-to-capacity ratio for the movement exceeds 1.0, regardless of the control delay.

The LOS criteria for TWSC intersections are somewhat different from the criteria used in Chapter 18 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, which can reduce users' delay tolerance.

Exhibit 19-1
Level-of-Service Criteria: Automobile Mode

Control Delay (s/vehicle)	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. LOS criteria for pedestrians are given in Exhibit 19-2.

Exhibit 19-2
Level-of-Service Criteria: Pedestrian Mode

LOS	Control Delay (s/pedestrian)	Comments
A	0–5	Usually no conflicting traffic
B	5–10	Occasionally some delay due to conflicting traffic
C	10–20	Delay noticeable to pedestrians, but not inconveniencing
D	20–30	Delay noticeable and irritating, increased likelihood of risk taking
E	30–45	Delay approaches tolerance level, risk-taking behavior likely
F	>45	Delay exceeds tolerance level, high likelihood of pedestrian risk taking

Note: Control delay may be interpreted as s/pedestrian group if groups of pedestrians were counted as opposed to individual pedestrians.

LOS F for pedestrians occurs when there are not enough gaps of suitable size to allow waiting pedestrians to cross through traffic on the major street safely. This situation is typically evident from extremely long control delays. The method is based on a constant critical headway. In the field, however, LOS F may also appear in the form of crossing pedestrians selecting smaller-than-usual gaps. In such cases, safety could be a concern that warrants further study.

REQUIRED INPUT DATA

Analysis of a TWSC intersection requires the following data:

1. Number and configuration of lanes on each approach;
2. Percentage of heavy vehicles for each movement;
3. Either of the following:
    a. Demand flow rate for each entering vehicular movement and each pedestrian crossing movement during the peak 15 min, or
    b. Demand flow rate for each entering vehicular movement and each pedestrian crossing movement during the peak hour and a peak hour factor for the hour;
4. Special geometric factors such as
    a. Unique channelization aspects,
    b. Existence of a two-way left-turn lane or raised or striped median storage (or both),

   c. Approach grades,
   d. Existence of flared approaches on the minor street, and
   e. Existence of upstream signals;

5. The rate at which motorists yield to pedestrians and the degree of pedestrian platooning (for pedestrian LOS analysis); and
6. Length of analysis period, generally a peak 15-min period within the peak hour.

SCOPE OF THE METHODOLOGY

This chapter focuses on TWSC intersection operations. This version of the TWSC intersection analysis procedures is primarily based on studies conducted by National Cooperative Highway Research Program Project 3-46 (*1*).

LIMITATIONS OF THE METHODOLOGY

Automobile Mode

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 17, Urban Street Segments. The methodologies do not apply to TWSC intersections with more than four approaches.

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 later in this chapter.

Pedestrian Mode

The limitations of the pedestrian methodologies are somewhat different from those of the automobile mode, as the methods were developed in separate research efforts. In this chapter, pedestrian methodologies apply to TWSC intersections and midblock crossings where pedestrians cross up to four through lanes on the major street. The analysis procedure does not apply to undivided streets with more than four lanes, although it can accommodate up to four lanes in each direction separated by a median. The methodologies do not account for interaction effects of upstream signalized intersections. The analysis procedure assumes random arrivals on the major street and equal directional and lane distribution on the major street. It does not account for the effects of upstream signals.

The analysis procedure does not take into account pedestrian cross-flows (i.e., pedestrian flows approximately perpendicular to and crossing another pedestrian stream) and assumes that 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.

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.

Bicycle Mode

At the time of publication of this edition of the HCM, the current methodologies for analyzing LOS and delay at TWSC intersections apply to bicycles in limited situations that are not supported by research. As such, there are no established LOS standards for bicycles at TWSC intersections. Additional research on bicycle behavior and operations at TWSC intersections needs to be done before procedures that adequately address these issues can be developed. A discussion of qualitative effects is included in the methodology section of this chapter.

2. METHODOLOGY

OVERVIEW

TWSC intersections require only drivers on the minor-street approaches to stop before proceeding into the intersection. Left-turning drivers from the major street may have to yield to oncoming major-street through or right-turning traffic but are not required to stop in the absence of oncoming traffic.

The methodologies presented rely on the required input data listed previously 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 use some time to move into 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.

The basic capacity model assumes that gaps in the conflicting movements are randomly distributed. When traffic signals on the major street are within 0.25 mi of the subject intersection, flows may not be random but will likely have some platoon structure.

For the automobile mode analysis, the methodology addresses a number of 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.

THEORETICAL BASIS

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 to consider is the proportion of gaps of a particular size on the major street offered to the driver entering from the minor street as well as the pattern of arrival times of vehicles. The distribution of gaps between the vehicles in the different streams has a major effect on the performance of the intersection.

Usefulness of Gaps

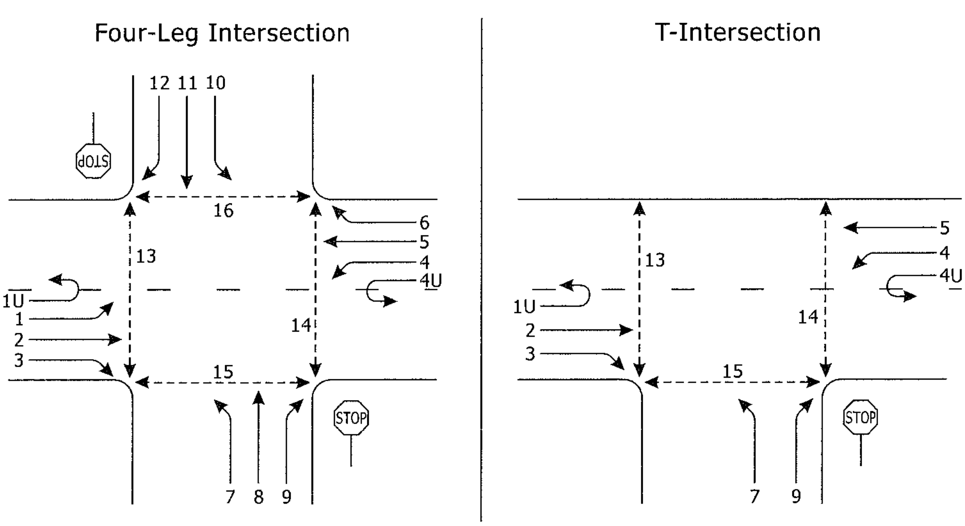
The second element to consider is the extent to which drivers find gaps of a particular size useful when they attempt to enter the intersection. It is generally assumed in gap-acceptance theory that drivers are both consistent and homogeneous. This assumption is not entirely correct. Studies have demonstrated that different drivers have different gap-acceptance thresholds and even 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

Each movement has a different ranking in a priority hierarchy. The gap-acceptance process evaluates them with impedance terms through the order of departures. Typically, gap-acceptance processes assume that drivers on the major street are unaffected by the minor-street movements. If this assumption is not the case, 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, while others have to give way or yield to higher-order movements. Exhibit 19-3 shows the assumed numbering of movements at both T- and four-leg intersections.

Exhibit 19-3
Vehicular and Pedestrian Movements at a TWSC Intersection



Movements can be categorized by right-of-way priority as follows:

- 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.

As an example of application of the priority of right-of-way, 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.

The *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.

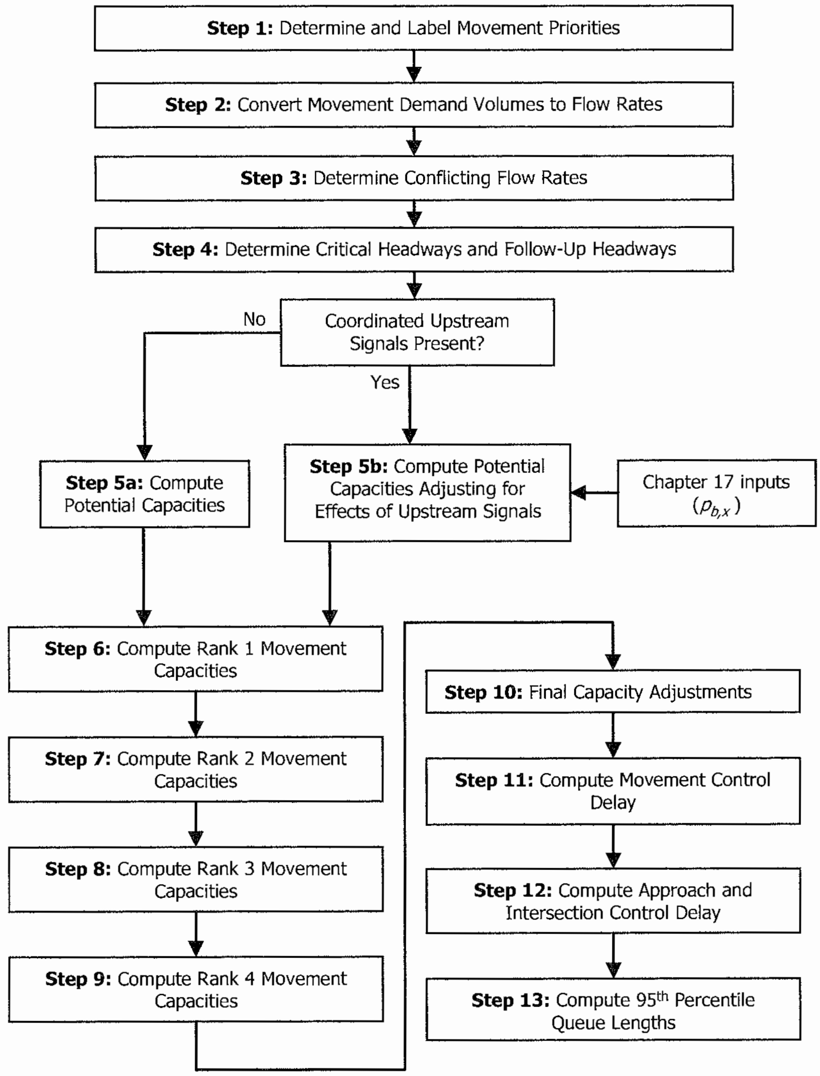
Follow-up headway defined.

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.

AUTOMOBILE MODE

The TWSC intersection methodology for the automobile 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 and incorporation of adjustments to compute movement capacities, and estimation of control delays and queue lengths. These steps are illustrated in Exhibit 19-4.

Exhibit 19-4
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. The process of this step also identifies for the analyst the sequence in which capacity computations will be completed. Because the methodology is based on prioritized use of gaps by vehicles at a TWSC intersection, it is important that the subsequent computations in the automobile mode 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 where 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 4.

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 19-1, through use of the peak hour factor for the intersection.

Equation 19-1

$$v_i = \frac{V_i}{PHF}$$

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.

Step 3: Determine Conflicting Flow Rates

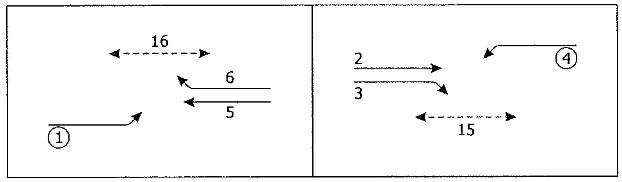
Each movement at a TWSC intersection faces a different set of conflicts that are directly related to the nature of the subject movement. The following subsections provide an illustration of the set of conflicts facing each minor movement (Rank 2 through Rank 4) at a TWSC intersection. These exhibits 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 (veh/h)] that conflicts with movement x.

Pedestrians may also conflict with vehicular movements. Pedestrian flow rates, also 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 it recognizes some peculiarities associated with pedestrian movements, this method takes a uniform approach to vehicular and pedestrian movements.

Major-Street Left-Turn Movements (Rank 2—Movements 1 and 4)

Exhibit 19-5 illustrates the conflicting movements, while Equation 19-2 and Equation 19-3 compute the conflicting flow encountered by major-street left-turning drivers. The left-turn movement from the major street is in conflict with the total opposing through and right-turn flow, because those vehicles must cross the opposing through movement and merge 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 turns from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

Exhibit 19-5
Definition of Conflicting Movements for Major-Street Left-Turn Movements



Equation 19-2

$$v_{c,1} = v_5 + v_6 + v_{16}$$

Equation 19-3

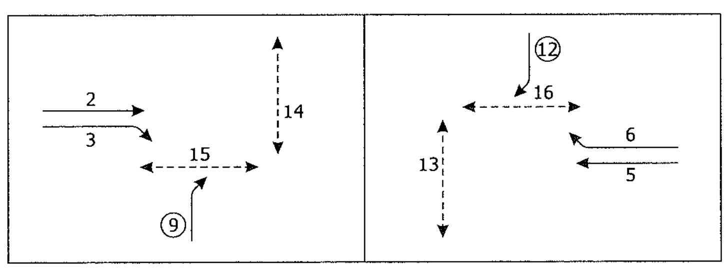
$$v_{c,4} = v_2 + v_3 + v_{15}$$

If the major-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, the v_6 and v_3 terms in Equation 19-2 and Equation 19-3, respectively, may be assumed to be zero.

Minor-Street Right-Turn Movements (Rank 2—Movements 9 and 12)

Exhibit 19-6 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 where 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 turn). 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 19-6 does not include major-street U-turns as conflicting vehicles. While these conflicts may be observed in practice, they are not assumed to be conflicts in this methodology.

Exhibit 19-6
Definition of Conflicting Movements for Minor-Street Right-Turn Movements



Equation 19-4 and Equation 19-5 compute the conflicting flow rate for minor-street right-turn movements entering two-lane major streets, Equation 19-6 and Equation 19-7 are used for four-lane major streets, and Equation 19-8 and Equation 19-9 are used for six-lane major streets. 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.

Two-lane major streets:

$$v_{c,9} = v_2 + 0.5v_3 + v_{14} + v_{15}$$ **Equation 19-4**

$$v_{c,12} = v_5 + 0.5v_6 + v_{13} + v_{16}$$ **Equation 19-5**

Four-lane major streets:

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15}$$ **Equation 19-6**

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{13} + v_{16}$$ **Equation 19-7**

Six-lane major streets:

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15}$$ **Equation 19-8**

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{13} + v_{16}$$ **Equation 19-9**

Major-Street U-Turn Movements (Rank 2—Movements 1U and 4U)

Exhibit 19-7 illustrates the conflicting movements encountered by major-street U-turning drivers. The U-turn movement from the major street is in conflict with the total opposing through and right-turn flow, similar to the major-street left-turn movement. Research found that the presence of minor-street right-turning vehicles significantly affects the capacity of major-street U-turns (5). The methodology accounts for this effect in the impedance calculation rather than here 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.

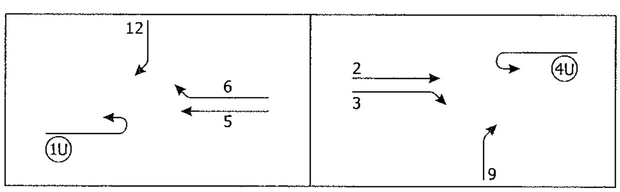


Exhibit 19-7
Definition of Conflicting Movements for Major-Street U-Turn Movements

Equation 19-10 and Equation 19-11 compute the conflicting flow rates for major-street U-turns, where the major street has four lanes. Equation 19-12 and Equation 19-13 compute the conflicting flow rates for major-street U-turns on six-lane major streets. (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.

Four-lane major streets:

$$v_{c,1U} = v_5 + v_6$$ **Equation 19-10**

$$v_{c,4U} = v_2 + v_3$$ **Equation 19-11**

Six-lane major streets:

$$v_{c,1U} = 0.73v_5 + 0.73v_6$$ **Equation 19-12**

$$v_{c,4U} = 0.73v_2 + 0.73v_3$$ **Equation 19-13**

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.

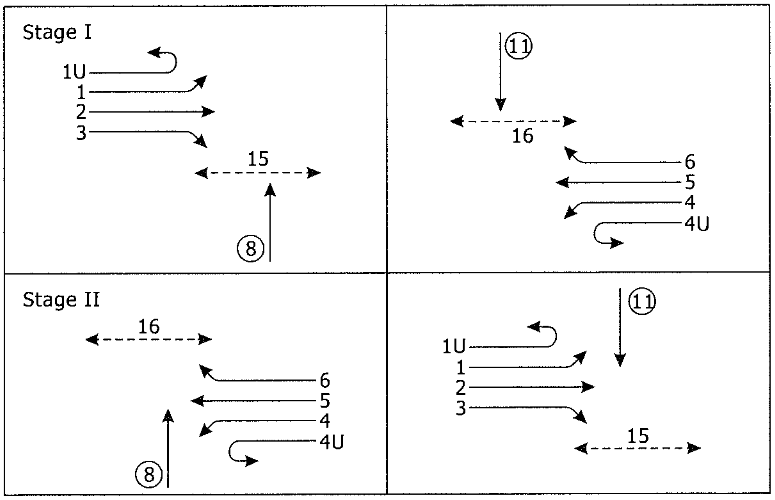
Minor-Street Through Movements (Rank 3—Movements 8 and 11)

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.

Minor-street through movements may complete their maneuver in one or two stages. Single-stage gap acceptance assumes no median refuge area is available for minor-street drivers to store in and that the minor-street drivers will be evaluating gaps in both major-street directions simultaneously. Conversely, the two-stage gap-acceptance scenario assumes that a median refuge area is available for minor-street drivers. During Stage I, minor-street drivers evaluate major-street gaps in the near-side traffic stream (conflicting traffic from the left); during Stage II, minor-street drivers evaluate major-street gaps in the far-side 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 19-8 illustrates the conflicting movements encountered by minor-street through-movement drivers.

Exhibit 19-8
Definition of Conflicting Movements for Minor-Street Through Movements



Equation 19-14 and Equation 19-15 compute the conflicting flow 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.

$$v_{c,I,8} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

Equation 19-14

$$v_{c,I,11} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$

Equation 19-15

Equation 19-16 and Equation 19-17 compute the conflicting flow 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.

$$v_{c,II,8} = 2(v_4 + v_{4U}) + v_5 + v_6 + v_{16}$$

Equation 19-16

$$v_{c,II,11} = 2(v_1 + v_{1U}) + v_2 + v_3 + v_{15}$$

Equation 19-17

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 of 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 19-9 illustrates the conflicting movements encountered by minor-street left-turning drivers.

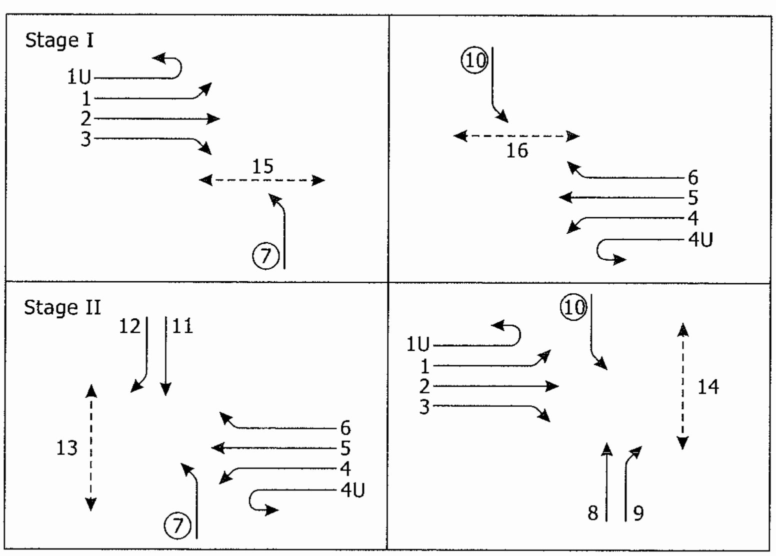


Exhibit 19-9
Conflicting Movements for Minor-Street Left-Turn Movements

During Stage I, Equation 19-18 and Equation 19-19 compute the conflicting flow rate for minor-street left-turn movements entering two-lane major streets, while Equation 19-20 and Equation 19-21 are used for four-lane major streets, and Equation 19-22 and Equation 19-23 are used for six-lane major streets. 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.

Two-lane major streets:

Equation 19-18

$$v_{c,I,7} = 2v_1 + v_2 + 0.5v_3 + v_{15}$$

Equation 19-19

$$v_{c,I,10} = 2v_4 + v_5 + 0.5v_6 + v_{16}$$

Four-lane major streets:

Equation 19-20

$$v_{c,I,7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

Equation 19-21

$$v_{c,I,10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$

Six-lane major streets:

Equation 19-22

$$v_{c,I,7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

Equation 19-23

$$v_{c,I,10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$

During Stage II, Equation 19-24 and Equation 19-25 compute the conflicting flow rate for minor-street left-turn movements entering two-lane major streets, while Equation 19-26 and Equation 19-27 are used for four-lane major streets, and Equation 19-28 and Equation 19-29 are used for six-lane major streets. 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.

Two-lane major streets:

Equation 19-24

$$v_{c,II,7} = 2v_4 + v_5 + 0.5v_6 + 0.5v_{12} + 0.5v_{11} + v_{13}$$

Equation 19-25

$$v_{c,II,10} = 2v_1 + v_2 + 0.5v_3 + 0.5v_9 + 0.5v_8 + v_{14}$$

Four-lane major streets:

Equation 19-26

$$v_{c,II,7} = 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$

Equation 19-27

$$v_{c,II,10} = 2(v_1 + v_{1U}) + 0.5v_2 + 0.5v_8 + v_{14}$$

Six-lane major streets:

Equation 19-28

$$v_{c,II,7} = 2(v_4 + v_{4U}) + 0.4v_5 + 0.5v_{11} + v_{13}$$

Equation 19-29

$$v_{c,II,10} = 2(v_1 + v_{1U}) + 0.4v_2 + 0.5v_8 + v_{14}$$

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 19-10 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 19-30:

Equation 19-30

$$t_{c,x} = t_{c,base} + t_{c,HV} P_{HV} + t_{c,G} G - t_{3,LT}$$

where

$t_{c,x}$ = critical headway for movement x (s);

$t_{c,base}$ = base critical headway from Exhibit 19-10 (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 (0.1 for Movements 9 and 12; 0.2 for Movements 7, 8, 10, and 11) (s);

G = percent grade (expressed as an integer; e.g., G = −2 for a 2% downhill grade); and

$t_{3,LT}$ = adjustment factor for intersection geometry (0.7 for minor-street left-turn movement at three-leg intersections; 0.0 otherwise) (s).

$t_{3,LT}$ is applicable to Movements 7, 8, 10, and 11

Exhibit 19-10
Base Critical Headways for TWSC Intersections

Vehicle Movement	Base Critical Headway, $t_{c,base}$ (s) Two Lanes	Four Lanes	Six Lanes
Left turn from major	4.1	4.1	5.3
U-turn from major	N/A	6.4 (wide) 6.9 (narrow)	5.6
Right turn from minor	6.2	6.9	7.1
Through traffic on minor	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* 2-stage, Stage I: 5.5* 2-stage, Stage II: 5.5*
Left turn from minor	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

* 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 far-side major-street through stream pass. 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 19-11. 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 19-31.

Equation 19-31

$$t_{f,x} = t_{f,base} + t_{f,HV} P_{HV}$$

where

$t_{f,x}$ = follow-up headway for movement x (s),

$t_{f,base}$ = base follow-up headway from Exhibit 19-11 (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 19-11
Base Follow-Up Headways for TWSC Intersections

	Base Follow-Up Headway, $t_{f,base}$ (s)		
Vehicle Movement	**Two Lanes**	**Four Lanes**	**Six Lanes**
Left turn from major	2.2	2.2	3.1
U-turn from major	N/A	2.5 (wide) 3.1 (narrow)	2.3
Right turn from minor	3.3	3.3	3.9
Through traffic on minor	4.0	4.0	4.0
Left turn from minor	3.5	3.5	3.8

Values from Exhibit 19-10 and Exhibit 19-11 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 If No Upstream Signal Effects Are Present

The potential capacity $c_{p,x}$ of a movement is computed according to the gap-acceptance model provided in Equation 19-32 (*6*). This model requires the analyst to input the conflicting flow rate $v_{c,x}$, the critical headway $t_{c,x}$, and the follow-up headway $t_{f,x}$, for movement x.

Equation 19-32

$$c_{p,x} = v_{c,x} \frac{e^{-v_{c,x} t_{c,x} / 3{,}600}}{1 - e^{-v_{c,x} t_{f,x} / 3{,}600}}$$

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 4 movements, the potential capacity is computed three times: $c_{p,x}$ assuming one-stage operation, $c_{p,I,x}$ for Stage I, and $c_{p,II,x}$ for Stage II. The conflicting flow definitions for each calculation are as provided in Step 4.

Step 5b: Potential Capacity If Upstream Signal Effects Are Present

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 19-3.

With these values, the proportion of the analysis period that is blocked for each minor movement can be computed by using Exhibit 19-12:

Exhibit 19-12
Proportion of Analysis Period Blocked for Each Movement

Movement(s) x	Proportion Blocked for Movement, $p_{b,x}$: One-Stage Movements	Two-Stage Movements: Stage I	Two-Stage Movements: Stage II
1, 1U	$p_{b,1}$	N/A	N/A
4, 4U	$p_{b,4}$	N/A	N/A
7	$p_{b,7}$	$p_{b,4}$	$p_{b,1}$
8	$p_{b,8}$	$p_{b,4}$	$p_{b,1}$
9	$p_{b,9}$	N/A	N/A
10	$p_{b,10}$	$p_{b,1}$	$p_{b,4}$
11	$p_{b,11}$	$p_{b,1}$	$p_{b,4}$
12	$p_{b,12}$	N/A	N/A

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 (*7*).

The conflicting flow for movement x during the unblocked period is given by Equation 19-33:

Equation 19-33

$$v_{c,u,x} = \begin{cases} \dfrac{v_{c,x} - 1.5 v_{c,\min} p_{b,x}}{1 - p_{b,x}} & \text{if } v_{c,x} > 1.5 v_{c,\min} p_{b,x} \\ 0 & \text{otherwise} \end{cases}$$

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 19-12.

The potential capacity of the subject movement x, accounting for the effect of platooning, is given by Equation 19-34 and Equation 19-35:

Equation 19-34

$$c_{p,x} = (1 - p_{b,x})c_{r,x}$$

Equation 19-35

$$c_{r,x} = v_{c,u,x} \frac{e^{-v_{c,u,x}t_{c,x}/3,600}}{1 - e^{-v_{c,u,x}t_{f,x}/3,600}}$$

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.

This equation uses the same critical headway and follow-up headway inputs as does a normal calculation but uses only the conflicting flow during the unblocked period.

Steps 6–9: Compute Movement Capacities

For clarity, these steps assume that pedestrian impedance effects can be neglected, and in many cases this is a reasonable assumption. However, pedestrians can be accounted for in the analysis of the automobile mode by replacing these steps with those provided in Chapter 32, STOP-Controlled Intersections: Supplemental, that incorporate the effects of pedestrian impedance.

Step 6: Rank 1 Movement Capacity

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: Rank 2 Movement Capacity

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 Movement

The movement capacity of each Rank 2 major-street left-turn movement (1 and 4) is equal to its potential capacity, as shown in Equation 19-36.

Equation 19-36

$$c_{m,j} = c_{p,j}$$

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, as shown in Equation 19-37.

Equation 19-37

$$c_{m,j} = c_{p,j}$$

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 19-38 and Equation 19-39, respectively.

$$f_{1U} = p_{0,12} = 1 - \frac{v_{12}}{c_{m,12}}$$

Equation 19-38

$$f_{4U} = p_{0,9} = 1 - \frac{v_9}{c_{m,9}}$$

Equation 19-39

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 19-40:

$$c_{m,jU} = (c_{p,jU}) f_{jU}$$

Equation 19-40

In almost all cases, major-street left-turning vehicles share a lane with U-turning vehicles. Therefore, if Rank 2 major-street U-turn movements are present to a significant degree, then Equation 19-59 should be used to compute the shared-lane capacity.

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.

Since the left-turn and U-turn movements are typically conducted from the same lane, their shared-lane capacity is computed with Equation 19-41:

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}} \right)}$$

Equation 19-41

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 19-42:

Equation 19-42

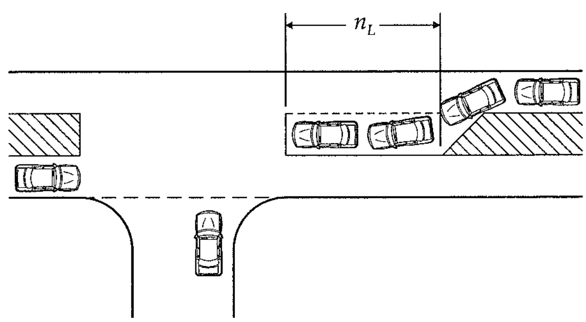
$$p_{0,j} = 1 - \frac{v_j}{c_{m,j}}$$

If major-street through and left-turn movements are shared, use Equation 19-43. Also, use Equation 19-42 to compute the probability of a queue-free state for Rank 3 movements.

where j = 1 and 4 (major-street left-turn and U-turn movements of Rank 2, using shared volume and capacity as appropriate).

If, however, a shared left-turn lane or a short left-turn pocket is present on a major-street approach (as in Exhibit 19-13), 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 19-43. This probability is then used by the analyst in lieu of $p_{0,j}$ (as computed by Equation 19-42).

Exhibit 19-13
Short Left-Turn Pocket



The methodology implicitly assumes that 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 19-43 and Equation 19-44 as an indication of the probability that there will be no queue in the respective major-street shared or short lanes (*8*).

Equation 19-43

$$p^*_{0,j} = 1 - \left(1 - p_{0,j}\right)\left[\sqrt[(n_L+1)]{1 + \frac{x_{i,1+2}^{(n_L+1)}}{1 - x_{i,1+2}}} \right]$$

Equation 19-44

$$x_{i,1+2} = \frac{v_{i1}}{s_{i1}} + \frac{v_{i2}}{s_{i2}}$$

where

$p_{0,j}$ = probability of queue-free state for movement j assuming an exclusive left-turn lane on the major street (per Equation 19-42);

j = 1 and 4 (major-street left-turning vehicular movements);

$i1$ = 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 = storage places in the left-turn pocket (see Exhibit 19-13).

When j = 1, $i1$ = 2 and $i2$ = 3; when j = 4, $i1$ = 5 and $i2$ = 6.

For the special situation of shared lanes ($n_L = 0$), Equation 19-43 becomes Equation 19-45 as follows:

$$p^*_{0,j} = 1 - \frac{1 - p_{0,j}}{1 - x_{i,1+2}}$$

Equation 19-45

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 19-42), the potential for queues on a major street with shared or short left-turn lanes may be taken into account.

Step 8: Compute Movement Capacities for Rank 3 Movements

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 Movement 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 19-46:

Equation 19-46

$$f_k = \prod_j p_{0,j}$$

where

$p_{0,j}$ = probability that conflicting Rank 2 movement j will operate in a queue-free state, and

k = Rank 3 movements.

The movement capacity for Rank 3 minor-street movements is computed with Equation 19-47, where f_k is the capacity adjustment factor that accounts for the impeding effects of higher-ranked movements computed according to Equation 19-46.

Equation 19-47

$$c_{m,k} = (c_{p,k}) f_k$$

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 19-48 and Equation 19-49, respectively.

Equation 19-48

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} \quad \text{for } n_m > 0$$

Equation 19-49

$$y = \frac{c_I - c_{m,x}}{c_{II} - v_L - c_{m,x}}$$

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, considering the total conflicting flow rate for both stages of a two-stage gap-acceptance process (from Step 8a).

The total capacity c_T for the subject movement, considering the two-stage gap-acceptance process, is computed by using Equation 19-50 and Equation 19-51 and incorporating the adjustment factors derived from Equation 19-48 and Equation 19-49.

For $y \neq 1$:

$$c_T = \frac{a}{y^{n_m+1}-1}\left[y\left(y^{n_m}-1\right)\left(c_{II}-v_L\right)+\left(y-1\right)c_{m,x}\right]$$

Equation 19-50

For $y = 1$:

$$c_T = \frac{a}{n_m+1}\left[n_m\left(c_{II}-v_L\right)+c_{m,x}\right]$$

Equation 19-51

Step 9: Compute Movement Capacities for Rank 4 Movements

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. At the same time, it must be recognized that 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 19-14 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 19-52.

$$p' = 0.65p'' - \frac{p''}{p''+3} + 0.6\sqrt{p''}$$

Equation 19-52

where

p' = adjustment to the major-street left, minor-street through impedance factor;

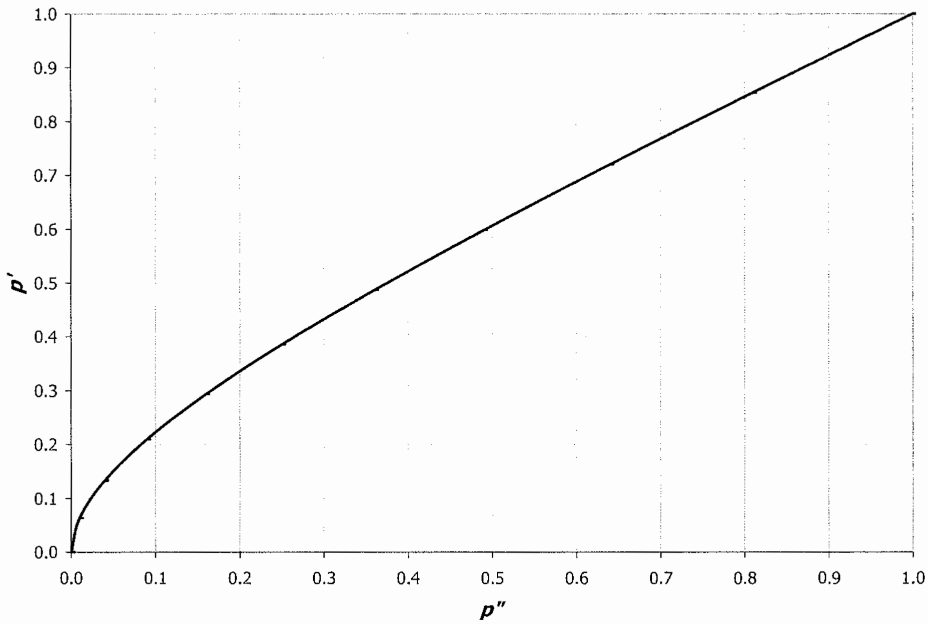
p'' = $(p_{0,j})(p_{0,k})$;

$p_{0,j}$ = 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 19-53, $p'' = (p_{0,1})(p_{0,4})(p_{0,11})$. Likewise, when determining p' for Rank 4 Movement 10, $p'' = (p_{0,1})(p_{0,4})(p_{0,8})$.

Exhibit 19-14
Adjustment to Impedance Factors for Major Left-Turn Movement and Minor 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 for the Rank 4 minor-street left-turn movement can be computed with Equation 19-53:

Equation 19-53

$$f_{p,l} = (p')(p_{0,j})$$

where

l = minor-street left-turn movement of Rank 4 (Movements 7 and 10 in Exhibit 19-3), and

j = conflicting Rank 2 minor-street right-turn movement (Movements 9 and 12 in Exhibit 19-3).

Finally, the movement capacity for the minor-street left-turn movements of Rank 4 is determined with Equation 19-54, where $f_{p,l}$ is the capacity adjustment factor that accounts for the impeding effects of higher-ranked movements.

Equation 19-54

$$c_{m,l} = (c_{p,l})f_{p,l}$$

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 19-55 and Equation 19-56, respectively:

Equation 19-55

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} \quad \text{for } n_m > 0$$

Equation 19-56

$$y = \frac{c_I - c_{m,x}}{c_{II} - v_L - c_{m,x}}$$

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 total capacity c_T for the subject movement considering the two-stage gap-acceptance process is computed by using Equation 19-57 and Equation 19-58 and incorporating the adjustment factors computed in Equation 19-55 and Equation 19-56.

For $y \neq 1$:

$$c_T = \frac{a}{y^{n_m+1} - 1}\left[y\left(y^{n_m} - 1\right)\left(c_{II} - v_L\right) + \left(y - 1\right)c_{m,x}\right]$$

Equation 19-57

For $y = 1$:

$$c_T = \frac{a}{n_m + 1}\left[n_m\left(c_{II} - v_L\right) + c_{m,x}\right]$$

Equation 19-58

Step 10: Final Capacity Adjustments

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

Where several movements share the same lane and cannot stop side by side at the stop line, Equation 19-59 is used to compute shared-lane capacity:

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)}$$

Equation 19-59

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: Compute Flared Minor-Street Lane Effects

To estimate the capacity of a flared right-turn lane (as in Exhibit 19-15), the average queue length for each movement sharing the right lane on the minor-street approach must first be computed with Equation 19-60. This computation assumes that the right-turn movement operates in one lane and that the other traffic in the right lane (upstream of the flare) operates in another, separate lane.

Equation 19-60

$$Q_{sep} = \frac{d_{sep} v_{sep}}{3{,}600}$$

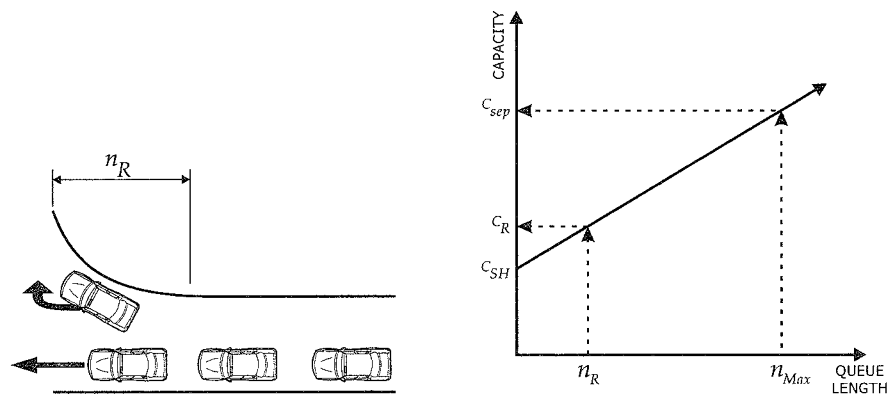
where

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).

Exhibit 19-15
Capacity of a Flared-Lane Approach



Next, the required length of the storage area such that the approach would operate effectively as separate lanes is computed with Equation 19-61. This is the maximum value of the queue lengths computed for each separate movement plus one vehicle.

Equation 19-61

$$n_{Max} = \operatorname*{Max}_{i}\left[\text{round}\left(Q_{sep,i} + 1\right)\right]$$

where

$Q_{sep,i}$ = average queue length for movement i considered as a separate lane;

round = round-off operator, rounding the quantity in parentheses to the nearest integer; and

n_{Max} = length of the storage area such that the approach would operate as separate lanes.

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 19-62, as shown:

Equation 19-62

$$c_{sep} = \text{Min}\left[c_R\left(1 + \frac{v_{L+TH}}{v_R}\right), c_{L+TH}\left(1 + \frac{v_R}{v_{L+TH}}\right)\right]$$

where

c_{sep} = sum of the capacities 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 19-15. A straight line is established by using 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 19-63.

$$c_R = \begin{cases} \left(c_{sep} - c_{SH}\right)\dfrac{n_R}{n_{Max}} + c_{SH} & \text{if } n_R \le n_{Max} \\ c_{sep} & \text{if } n_R > n_{Max} \end{cases}$$

Equation 19-63

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 19-62) (veh/h),

c_{SH} = capacity of the lane when all traffic is sharing one lane (veh/h), and

n_R = actual storage area for right-turning vehicles as defined in Exhibit 19-15.

The actual capacity c_{act} 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 made up of a number of factors that relate to 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 vehicles from free-flow speed to the speed of vehicles in 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 19-64) assumes that 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 19-64

$$d=\frac{3600}{c_{m,x}}+900T\left[\frac{v_x}{c_{m,x}}-1+\sqrt{\left(\frac{v_x}{c_{m,x}}-1\right)^2+\frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right]+5$$

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 (equals 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 19-64 to account for the deceleration of vehicles from free-flow speed to the speed of vehicles in 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 sneak by or bypass the left-turning vehicle. 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 19-43 ($p^*_{0,j}$ should be substituted for the major left-turn factor $p_{0,j}$ in Equation 19-43 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 19-65.

$$d_{Rank1} = \begin{cases} \dfrac{\left(1-p^{*}_{0,j}\right) d_{M,LT} \left(\dfrac{v_{i,1}}{N}\right)}{v_{i,1}+v_{i,2}} & N>1 \\ \\ \left(1-p^{*}_{0,j}\right) d_{M,LT} & N=1 \end{cases}$$

Equation 19-65

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 19-43;

$d_{M,LT}$ = delay to major left-turning vehicles, from Equation 19-64 (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 that 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 19-66 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 19-66

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 can be computed with

$$d_I = \frac{d_{A,1} v_{A,1} + d_{A,2} v_{A,2} + d_{A,3} v_{A,3} + d_{A,4} v_{A,4}}{v_{A,1} + v_{A,2} + v_{A,3} + v_{A,4}}$$

Equation 19-67

where

$d_{A,x}$ = control delay on approach x (s/veh), and

$v_{A,x}$ = volume or flow rate on approach x (veh/h).

In applying Equation 19-66 and Equation 19-67, 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 19-68 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 as follows (*9*):

Equation 19-68

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\frac{c_{m,x}}{3{,}600}\right)$$

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

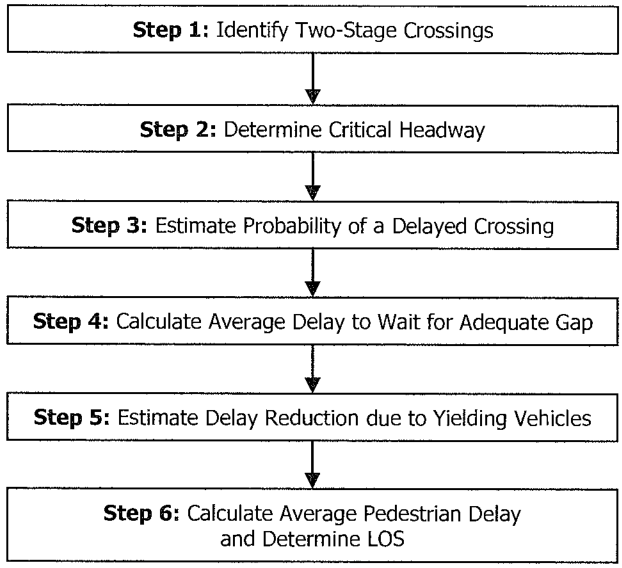
T = analysis time period (0.25 h for a 15-min period) (h).

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.

PEDESTRIAN MODE

The TWSC intersection methodology for the pedestrian mode is applied through a series of steps requiring input data related to vehicle and pedestrian volumes, geometric conditions, and motorist yield rates to pedestrians. These data are used to calculate the average pedestrian delay associated with pedestrian crossings of unsignalized and non-STOP-controlled roadways. The required steps are illustrated in Exhibit 19-16.

Exhibit 19-16
TWSC Pedestrian Methodology



Step 1: Identify Two-Stage Crossings

When a raised pedestrian-median refuge island is available, pedestrians typically cross in two stages, similar to the two-stage gap-acceptance described for automobiles earlier in this chapter. Determination of whether a pedestrian-median refuge exists may require engineering judgment. 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 to 6. To determine pedestrian LOS, the pedestrian delay for each stage should be summed to establish the average pedestrian delay associated with the entire crossing. This service measure is used to determine pedestrian LOS for a TWSC intersection with two-stage crossings.

Step 2: Determine Critical Headway

Critical headway for pedestrians.

The procedure for estimating the critical headway is similar to that described for automobiles. The critical headway is the time 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 that the pedestrian will cross, but if the available headway is less than the critical headway, it is assumed that the pedestrian will not cross.

For a single pedestrian, critical headway is computed with Equation 19-69:

$$t_c = \frac{L}{S_p} + t_s$$

Equation 19-69

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).

Pedestrian platooning.

If pedestrian platooning is observed in the field, then the spatial distribution of pedestrians should be computed with Equation 19-70. If no platooning is observed, the spatial distribution of pedestrians is assumed to be 1.

Equation 19-70

$$N_p = \mathrm{Int}\left[\frac{8.0(N_c - 1)}{W_c}\right] + 1$$

where

N_p = spatial distribution of pedestrians (ped);

N_c = total number of pedestrians in the crossing platoon, from Equation 19-71 (ped);

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 19-71:

Equation 19-71

$$N_c = \frac{v_p e^{v_p t_c} + v e^{-v t_c}}{(v_p + v) e^{(v_p - v) t_c}}$$

where

N_c = total number of pedestrians in the crossing platoon (ped),

v_p = pedestrian flow rate (ped/s),

v = vehicular flow rate (veh/s), and

t_c = single pedestrian critical headway (s).

Group critical headway is determined with Equation 19-72:

Equation 19-72

$$t_{c,G} = t_c + 2(N_p - 1)$$

where

$t_{c,G}$ = group critical headway (s),

t_c = critical headway for a single pedestrian (s), and

N_p = spatial distribution of pedestrians (ped).

Step 3: Estimate Probability of a Delayed Crossing

On the basis of calculation of the critical headway t_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 probability of encountering a headway exceeding the critical headway in any given lane can be estimated by using a Poisson distribution. The likelihood that a gap in a given lane does not exceed the critical headway is thus the complement as shown in Equation 19-73. Because traffic is assumed to be distributed independently in each through lane, Equation 19-74 shows the probability that a pedestrian incurs nonzero delay at a TWSC crossing.

$$P_b = 1 - e^{\frac{-t_{c,G} v}{L}}$$

Equation 19-73

$$P_d = 1 - (1 - P_b)^L$$

Equation 19-74

where

P_b = probability of a blocked lane,

P_d = probability of a delayed crossing,

L = number of through lanes crossed,

$t_{c,G}$ = group critical headway (s), and

v = vehicular flow rate (veh/s).

Step 4: Calculate Average Delay to Wait for Adequate Gap

Research indicates that average delay to pedestrians at unsignalized crossings, assuming that 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 (*10*). The average delay per pedestrian to wait for an adequate gap is given by Equation 19-75.

$$d_g = \frac{1}{v}\left(e^{v t_{c,G}} - v t_{c,G} - 1\right)$$

Equation 19-75

where

d_g = average pedestrian gap delay (s),

$t_{c,G}$ = group critical headway (s), and

v = vehicular flow rate (veh/s).

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 19-76:

$$d_{gd} = \frac{d_g}{P_d}$$

Equation 19-76

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: Estimate Delay Reduction due to Yielding Vehicles

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 19-75 estimates pedestrian delay when motorists on the major approaches do not yield to pedestrians. Where motorist yield rates are significantly higher than zero, pedestrians will experience considerably less delay than that estimated by Equation 19-75.

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. Motorist yield rates are influenced by a range of factors, including roadway geometry, travel speeds, pedestrian crossing treatments, local culture, and law enforcement practices.

Research (*11, 12*) provides information on motorist responses to typical pedestrian crossing treatments, as shown in Exhibit 19-17. The exhibit shows results from two separate data collection methods. Staged data were collected with pedestrians trained by the research team to maintain consistent positioning, stance, and aggressiveness in crossing attempts. Unstaged data were collected through video recordings of the general population. The values shown in Exhibit 19-17 are based on a limited number of sites and do not encompass the full range of available crossing treatments. As always, practitioners should supplement these values with local knowledge and engineering judgment.

Exhibit 19-17
Effect of Pedestrian Crossing Treatments on Motorist Yield Rates

Crossing Treatment	Staged Pedestrians		Unstaged Pedestrians	
	Number of Sites	Mean Yield Rate, %	Number of Sites	Mean Yield Rate, %
Overhead flashing beacon (push button activation)	3	47	4	49
Overhead flashing beacon (passive activation)	3	31	3	67
Pedestrian crossing flags	6	65	4	74
In-street crossing signs (25–30 mi/h)	3	87	3	90
High-visibility signs and markings (35 mi/h)	2	17	2	20
High-visibility signs and markings (25 mi/h)	1	61	1	91
Rectangular rapid-flash beacon	N/A	N/A	17	81

Source: Fitzpatrick et al. (*11*) and Shurbutt et al. (*12*).

Depending on the crossing treatment and other factors, motorist behavior varies significantly.

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 through lanes on the major street. Consider a pedestrian waiting for a crossing opportunity at a TWSC intersection, with vehicles in each conflicting through lane arriving every h seconds. On average, a potential yielding event will occur every h seconds, where $P(Y)$ represents the probability of motorists yielding for a given event. As vehicles are assumed to arrive randomly, each potential yielding event is considered to be independent.

For any given 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. The pedestrian may cross only if vehicles in each blocked lane choose to yield.

If not, the pedestrian must wait an additional h seconds for the next 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.

Thus, average pedestrian delay can be calculated with Equation 19-77, 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 where pedestrians wait for an adequate gap.

Equation 19-77

$$d_p = \sum_{i=1}^{n} h(i-0.5)P(Y_i) + \left(P_d - \sum_{i=1}^{n} P(Y_i) \right) d_{gd}$$

where

d_p = average pedestrian delay (s),

i = crossing event ($i = 1$ to n),

h = average headway for each through lane,

$P(Y_i)$ = probability that motorists yield to pedestrian on crossing event i, and

n = Int(d_{gd}/h), average number of crossing events before an adequate gap is available.

Equation 19-77 requires the calculation of $P(Y_i)$. The probabilities $P(Y_i)$ that motorists will yield for a given crossing event are considered below for pedestrian crossings of one, two, three, and four through lanes.

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. For $i = 2$, $P(Y_i)$ is equal to M_y multiplied by the probability that the second yielding event occurs (i.e., that the pedestrian did not cross on the first yielding event), $P_d^*(1 - M_y)$. Equation 19-78 gives $P(Y_i)$ for any i.

Equation 19-78

$$P(Y_i) = P_d M_y (1 - M_y)^{i-1}$$

where

M_y = motorist yield rate (decimal), and

i = crossing event ($i = 1$ to n).

Two-Lane Crossing

For a two-lane pedestrian crossing at a TWSC intersection, $P(Y_i)$ 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 equal to Equation 19-79:

Equation 19-79

$$P(Y_1) = 2P_b(1 - P_b)M_y + P_b^{\,2}M_y^{\,2}$$

Equation 19-80 shows $P(Y_i)$ where i is greater than 1. Equation 19-80 is equivalent to Equation 19-79 if $P(Y_0)$ is set to equal 0.

Equation 19-80

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{(2P_b(1-P_b)M_y) + (P_b^2 M_y^2)}{P_d}\right]$$

Three-Lane Crossing

A three-lane crossing follows the same principles as a two-lane crossing. Equation 19-81 shows the calculation for $P(Y_i)$:

Equation 19-81

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right] \times \left[\frac{P_b^3 M_y^3 + 3P_b^2(1-P_b)M_y^2 + 3P_b(1-P_b)^2 M_y}{P_d}\right]$$

where $P(Y_0) = 0$.

Four-Lane Crossing

A four-lane crossing follows the same principles as above. Equation 19-82 shows the calculation for $P(Y_i)$:

Equation 19-82

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right] \times$$

$$\left[\frac{P_b^4 M_y^4 + 4P_b^3(1-P_b)M_y^3 + 6P_b^2(1-P_b)^2 M_y^2 + 4P_b(1-P_b^3)M_y}{P_d}\right]$$

where $P(Y_0) = 0$.

Step 6: Calculate Average Pedestrian Delay and Determine LOS

The delay experienced by a pedestrian is the service measure. Exhibit 19-2 lists LOS criteria for pedestrians at TWSC intersections based on pedestrian delay. Pedestrian delay at TWSC intersections with two-stage crossings is equal to the sum of the delay for each stage of the crossing.

BICYCLE MODE

As of the publication date 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, bicyclists may travel through the intersection either as a motor vehicle or as a pedestrian. Critical headway distributions have been identified in the research (*13, 14*) 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 fact 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.

3. APPLICATIONS

DEFAULT VALUES

A comprehensive presentation of potential default values for interrupted flow facilities is provided elsewhere (*15*), with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of *PHF* and percent heavy vehicles (%HV). 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.

The following general default values may be applied to a TWSC intersection analysis:

- $PHF = 0.92$
- %HV = 3

Additional default values are sometimes required. For the analysis of shared or short major-street left-turn lanes, the following assumed default values may be applied for the saturation flow rates of the major-street through and right-turn movements:

- Major-street through movement, s_{i1} = 1,800 veh/h
- Major-street right-turn movement, s_{i2} = 1,500 veh/h

For analysis of pedestrians at TWSC intersections, the following default values may be applied:

- Average pedestrian walking speed, S_p = 3.5 ft/s
- Pedestrian start-up time and end clearance time, t_s = 3 s

As the number of default values used in any analysis increases, its accuracy becomes more approximate, and the result may be significantly different from the actual outcome, depending on local conditions.

ESTABLISH INTERSECTION BOUNDARIES

This methodology assumes that the TWSC intersection under investigation is isolated, with the exception of a TWSC intersection that is located within 0.25 mi of a signalized intersection (for the major-street approaches). When interaction effects are likely between the subject TWSC intersection and other intersections (e.g., queue spillback, demand starvation), the use of alternative tools may result in 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.

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, %HV for each approach, *PHF* 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 %HV and *PHF* are typically estimated (or defaults are used) when planning applications are performed.

Interpreting 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* (MUTCD; *16*). This section discusses two common situations analysts face: the analysis of shared versus separate lanes and the interpretation of LOS F.

Interpretation of the effects of shared lanes should take into account 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. If left turns are placed in a shared lane, the control delay for vehicles in that 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.

PERFORMANCE MEASURES

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, resulting 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 an LOS F estimate, most low-volume minor-street approaches would not meet any of the MUTCD volume or delay warrants for signalization. As a result, analysts who use the HCM LOS thresholds 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 in addition to delay, such as volume-to-capacity (v/c) ratios for individual movements, average queue lengths, and 95th percentile queue lengths. 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.

USE OF ALTERNATIVE TOOLS

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 19-18, along with the potential for improved treatment by alternative tools.

Exhibit 19-18
Limitations of the HCM Signalized Intersection 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.
Macroscopic treatment of pedestrians and bicycles	Some simulation tools offer a microscopic modeling approach that provides explicit treatment of pedestrians and bicycles.

The most common application of alternative tools for TWSC involves an unsignalized intersection within a signalized arterial street.

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 main application for alternative tools at TWSC intersections involves coordinated arterial systems. Most intersections (i.e., those that are between the signals) 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. While 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.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The performance measure that determines LOS for unsignalized intersections is *control delay,* defined as that portion of the delay that is due to the existence of the 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(s) 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 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 24, Concepts: Supplemental, provides recommendations on how individual vehicle trajectories should be interpreted to produce specific performance measures. Of

Delay and LOS should be estimated only by using alternative tools that conform to these definitions and computations of queue delay presented in this manual.

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, the 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.

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, they are dealing in 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 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 Chapter 18, Signalized Intersections, guidance.

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 overriding logic that limits the amount of time any driver is willing to wait for a gap. The overriding logic is somewhat tool specific.

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 all the analysis tools.

Adjustment of Simulation Parameters to the HCM Parameters

The 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

It was mentioned previously that the most common application for TWSC simulation involves unsignalized intersections within a signalized arterial system. An example of this situation 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.

4. EXAMPLE PROBLEMS

Exhibit 19-19
List of Example Problems

Example Problem	Title	Type of Analysis
1	TWSC T-intersection	Operational analysis
2	TWSC pedestrian crossing	Operational analysis

EXAMPLE PROBLEM 1: TWSC T-INTERSECTION

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,
- Percent 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 19-20.

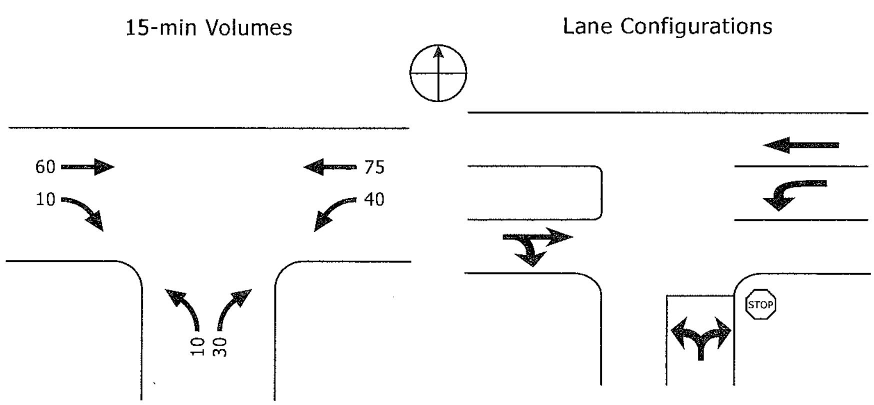


Exhibit 19-20
Example Problem 1 Movement Priorities, Lane Configurations, and Volumes

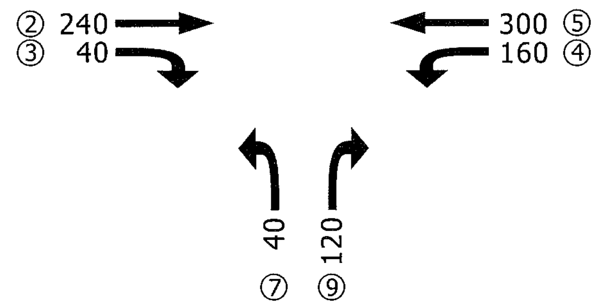
Comments

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 4 to determine a peak 15-min flow rate (in veh/h) for each movement. These values, along with the associated movement numbers, are shown in Exhibit 19-21.

Exhibit 19-21
Example Problem 1: 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 19-3, Equation 19-4, Equation 19-18, and Equation 19-24. The conflicting flow for the major-street left-turn $v_{c,4}$ is computed as follows:

$$v_{c,4} = v_2 + v_3 + v_{15}$$

$$v_{c,4} = 240 + 40 + 0 = 280 \text{ veh/h}$$

The conflicting flow for the minor-street right-turn movement $v_{c,9}$ is computed as follows:

$$v_{c,9} = v_2 + 0.5v_3 + v_{14} + v_{15}$$

$$v_{c,9} = 240 + 0.5(40) + 0 + 0 = 260 \text{ veh/h}$$

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 19-18) and Stage II (Equation 19-24) 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 19-10. The base critical headway for each movement is then adjusted according to Equation 19-30. The critical headway for the major-street left-turn $t_{c,4}$ is computed as follows:

$$t_{c,4} = t_{c,base} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

$$t_{c,4} = 4.1 + 1.0(0.1) + 0(0) - 0 = 4.2 \text{ s}$$

Similarly, the critical headway for the minor-street right-turn $t_{c,9}$ is computed as follows:

$$t_{c,9} = 6.2 + 1.0(0.1) + 0.1(0) - 0 = 6.3 \text{ s}$$

Finally, the critical headway for the minor-street left-turn $t_{c,7}$ is computed as follows:

$$t_{c,7} = 7.1 + 1.0(0.1) + 0.2(0) - 0.7 = 6.5 \text{ s}$$

The follow-up headway for each minor movement is computed beginning with the base follow-up headway given in Exhibit 19-11. The base follow-up headway for each movement is then adjusted according to Equation 19-31. The follow-up headway for the major-street left-turn $t_{f,4}$ 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 $t_{f,9}$ is computed as follows:

$$t_{f,9} = 3.3 + 0.9(0.1) = 3.39 \text{ s}$$

Finally, the follow-up headway for the minor-street left-turn $t_{f,7}$ is computed as follows:

$$t_{f,7} = 3.5 + 0.9(0.1) = 3.59 \text{ s}$$

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 $c_{p,4}$ is computed as follows:

$$c_{p,4} = v_{c,4} \frac{e^{-v_{c,4} t_{c,4} / 3{,}600}}{1 - e^{-v_{c,4} t_{f,4} / 3{,}600}} = 280 \frac{e^{-(280)(4.2)/3{,}600}}{1 - e^{-(280)(2.29)/3{,}600}} = 1{,}238 \text{ veh/h}$$

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 computed as follows:

$$c_{p,7} = 880 \frac{e^{-(880)(6.5)/3{,}600}}{1 - e^{-(880)(3.59)/3{,}600}} = 308 \text{ veh/h}$$

There are no upstream signals, so the adjustments for upstream signals are ignored.

Step 6: Compute Movement Capacities for Rank 1 Movements

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 Movement Capacities for Rank 2 Movements

The movement capacity for the major-street left-turn movement (Rank 2) $c_{m,4}$ is computed as follows:

$$c_{m,4} = (c_{p,4}) = 1{,}238 \text{ veh/h}$$

Similarly, the movement capacity for the minor-street right-turn movement (Rank 2) $c_{m,9}$ is computed as follows:

$$c_{m,9} = (c_{p,9}) = 760 \text{ veh/h}$$

Step 8: Compute Movement Capacities for Rank 3 Movements

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 operating in a queue-free state, $p_{0,4}$, is computed as follows:

$$p_{0,4} = 1 - \frac{v_4}{c_{m,4}} = 1 - \frac{160}{1{,}238} = 0.871$$

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 19-46 as follows:

$$f_7 = \prod_j p_{0,j} = 0.871$$

The movement capacity for the minor-street left-turn movement (Rank 3) $c_{m,7}$ is computed as follows:

$$c_{m,7} = (c_{p,7}) f_7 = (308)0.871 = 268 \text{ veh/h}$$

Step 9: Compute Movement Capacities for Rank 4 Movements

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 as follows:

$$c_{SH,NB} = \frac{\sum_y v_y}{\sum_y \left(\frac{v_y}{c_{m,y}}\right)} = \frac{v_7 + v_9}{\frac{v_7}{c_{m,7}} + \frac{v_9}{c_{m,9}}} = \frac{40 + 120}{\frac{40}{268} + \frac{120}{760}} = 521 \text{ veh/h}$$

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 as follows:

$$d_4 = \frac{3600}{c_{m,4}} + 900T\left[\frac{v_4}{c_{m,4}} - 1 + \sqrt{\left(\frac{v_4}{c_{m,4}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,4}}\right)\left(\frac{v_4}{c_{m,4}}\right)}{450T}}\right] + 5$$

$$d_4 = \frac{3{,}600}{1{,}238} + 900(.25)\left[\frac{160}{1{,}238} - 1 + \sqrt{\left(\frac{160}{1{,}238} - 1\right)^2 + \frac{\left(\frac{3{,}600}{1{,}238}\right)\left(\frac{160}{1{,}238}\right)}{450(0.25)}}\right] + 5 = 8.3 \text{ s}$$

On the basis of Exhibit 19-1, the westbound left-turn movement is assigned 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:

$$d_{SH,NB} = \frac{3{,}600}{521} + 900(0.25)\left[\frac{160}{521} - 1 + \sqrt{\left(\frac{160}{521} - 1\right)^2 + \frac{\left(\frac{3{,}600}{521}\right)\left(\frac{160}{521}\right)}{450(0.25)}}\right] + 5 = 14.9 \text{ s}$$

On the basis of Exhibit 19-1, 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 that the eastbound through movement v_2 and eastbound major-street right-turn movement v_3 do not incur any delay at this intersection.

Step 11c: 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 as follows:

$$d_{A,WB} = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

$$d_{A,WB} = \frac{0(0) + 0(300) + 8.3(160)}{0 + 300 + 160} = 2.9 \text{ s}$$

It is assumed that the westbound through movement incurs no control delay at this intersection. The control delay for the northbound approach was computed in Step 11a as $c_{SH,NB}$.

The intersection delay d_I is computed as follows:

$$d_I = \frac{d_{A,EB} v_{A,EB} + d_{A,WB} v_{A,WB} + d_{A,NB} v_{A,NB}}{v_{A,EB} + v_{A,WB} + v_{A,NB}}$$

$$d_I = \frac{0(280) + 2.9(460) + 14.9(160)}{280 + 460 + 160} = 4.1 \text{ s}$$

As noted previously, neither major-street approach LOS nor intersection LOS is defined.

Step 12: Compute 95th Percentile Queue Lengths

The 95th percentile queue length for the major-street westbound left-turn movement, $Q_{95,4}$, is computed as follows:

$$Q_{95,4} \approx 900T \left[\frac{v_4}{c_{m,4}} - 1 + \sqrt{\left(\frac{v_4}{c_{m,4}} - 1 \right)^2 + \frac{\left(\frac{3600}{c_{m,4}} \right)\left(\frac{v_4}{c_{m,4}} \right)}{150T}} \right] \left(\frac{c_{m,4}}{3600} \right)$$

$$Q_{95,4} \approx 900(0.25) \left[\frac{160}{1238} - 1 + \sqrt{\left(\frac{160}{1238} - 1 \right)^2 + \frac{\left(\frac{3600}{1238} \right)\left(\frac{160}{1238} \right)}{150(0.25)}} \right] \left(\frac{1238}{3600} \right) = 0.4 \text{ veh}$$

The result of 0.4 veh for the 95th percentile queue indicates that 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:

$$Q_{95,NB} \approx 900(0.25)\left[\frac{160}{521} - 1 + \sqrt{\left(\frac{160}{521} - 1\right)^2 + \frac{\left(\frac{3{,}600}{521}\right)\left(\frac{160}{521}\right)}{150(0.25)}}\right]\left(\frac{521}{3{,}600}\right) = 1.3 \text{ veh}$$

The result suggests that a queue of more than one vehicle will occur only occasionally for the northbound approach.

Discussion

Overall, the results indicate that the three-leg, TWSC intersection will operate well with small delays and little queuing for all minor movements.

EXAMPLE PROBLEM 2: TWSC PEDESTRIAN CROSSING

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: Unmarked crosswalk, median refuge island; and
- Scenario C: Marked crosswalk with high-visibility treatments, median refuge island.

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;
- Crosswalk length without median = 46 ft;
- Crosswalk length with median = 40 ft;
- Observed pedestrian walking speed = 4 ft/s;
- Pedestrian start-up time = 3 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 motor vehicle yield rates are 0% with unmarked crosswalks and 50% with high-visibility marked crosswalks.

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 equidistant pedestrian crossings of 20 ft to determine the total delay.

Step 2: Determine Critical Headway

Because there is no pedestrian platooning, the critical headway is determined with Equation 19-69:

Scenario A: $t_c = (46 \text{ ft}/4 \text{ ft/s}) + 3 \text{ s} = 14.5 \text{ s}$

Scenario B: $t_c = (20 \text{ ft}/4 \text{ ft/s}) + 3 \text{ s} = 8 \text{ s}$

Scenario C: $t_c = (20 \text{ ft}/4 \text{ ft/s}) + 3 \text{ s} = 8 \text{ s}$

Step 3: Estimate Probability of a Delayed Crossing

Equation 19-73 and Equation 19-74 are used to calculate P_b, the probability of a blocked lane, and P_d, the probability of a blocked 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.

$$P_b = 1 - e^{\frac{-t_{c,G} v}{L}}$$

$$P_d = 1 - (1 - P_b)^L$$

where

P_b = probability of a blocked lane,

P_d = probability of a delayed crossing,

L = number of through lanes crossed,

$t_{c,G}$ = group critical headway (s), and

v = vehicular flow rate (veh/s).

For the single-stage crossing, v is (1,700 veh/h)/(3,600 s/h) = 0.47 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) = 0.24 veh/s.

Scenario A:

$$P_b = 1 - e^{\frac{-14.5 \times 0.47}{4}} = 0.82$$

$$P_d = 1 - (0.18)^4 = 0.999$$

Scenario B:

$$P_b = 1 - e^{\frac{-8 \times 0.24}{2}} = 0.61$$

$$P_d = 1 - (0.39)^2 = 0.85$$

Scenario C:

$$P_b = 1 - e^{\frac{-8\times 0.24}{2}} = 0.61$$

$$P_d = 1 - (0.39)^2 = 0.85$$

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 19-75 and Equation 19-76.

Scenario A:

$$d_g = \frac{1}{0.47} \times \left(e^{0.47\times 14.5} - 0.47 \times 14.5 - 1\right) = 1{,}977 \text{ s}$$

$$d_{gd} = \frac{1{,}977}{0.999} = 1{,}979 \text{ s}$$

Scenario B:

$$d_g = \frac{1}{0.24}\left(e^{0.24\times 8} - 0.24 \times 8 - 1\right) = 15.8 \text{ s}$$

$$d_{gd} = \frac{15.8}{0.85} = 18.6 \text{ s}$$

Scenario C:

$$d_g = \frac{1}{0.24}\left(e^{0.24\times 8} - 0.24 \times 8 - 1\right) = 15.8 \text{ s}$$

$$d_{gd} = \frac{15.8}{0.85} = 18.6 \text{ s}$$

Step 5: Estimate Delay Reduction due to Yielding Vehicles

Under Scenarios A and B, the motorist yield rates are approximately 0%. Therefore, there is no reduction in delay due to yielding vehicles, and average delay is the same as that shown in Step 4. Under Scenario C, motorist yield rates are 50%. Because of the two-stage crossing, use Equation 19-80 to determine $P(Y_i)$:

$$P(Y_1) = [0.85 - 0]\left[\frac{(2 \times 0.61(1-0.61)0.50) + (0.61^2 0.50^2)}{0.85}\right] = 0.33$$

$$P(Y_2) = [0.85 - 0.33]\left[\frac{(2 \times 0.61(1-0.61)0.50) + (0.61^2 0.50^2)}{0.85}\right] = 0.20$$

The results of Equation 19-80 can be substituted into Equation 19-77 to determine average pedestrian delay.

$$d_p = \sum_{i=1}^{2} 8.5(i-0.5)P(Y_i) + \left(0.85 - \sum_{i=1}^{2} P(Y_i)\right)18.6 = 9.8\text{ s}$$

Step 6: Calculate LOS

Average pedestrian delays and pedestrian LOS under each of the three scenarios are as follows:

Scenario A = 1,979 s = LOS F

Scenario B = 2 × 15.8 s = 31.6 s = LOS E

Scenario C = 2 × 9.8 s = 19.6 s = LOS C

5. REFERENCES

Some of these references are available in the Technical Reference Library in Volume 4.

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CHAPTER 20
ALL-WAY STOP-CONTROLLED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

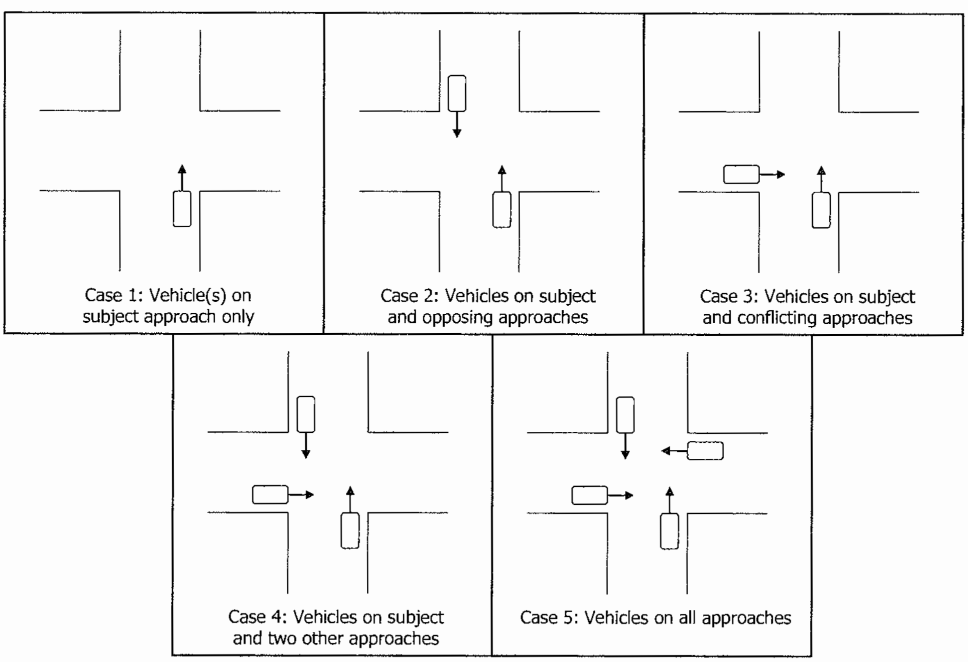
Chapter 20, All-Way STOP-Controlled Intersections, presents concepts and procedures for analyzing these types of intersections (*1*). A glossary and list of symbols, including those used for all-way STOP-controlled (AWSC) intersections, is provided in Chapter 9.

AWSC intersections require every vehicle to stop at the intersection before proceeding. Because each driver must stop, the decision to proceed into the intersection is a function of traffic conditions on the other approaches. If no traffic is present on the other approaches, a driver can proceed immediately after stopping. If there is traffic on one or more of the other approaches, a driver proceeds only after determining that no vehicles are currently in the intersection and that it is the driver's turn to proceed.

Field observations indicate that standard four-leg AWSC intersections operate in either a two-phase or a four-phase pattern, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north–south and east–west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach intersection).

If traffic is present on the subject approach only, vehicles depart as rapidly as individual drivers can safely accelerate into and clear the intersection. This case is illustrated as Case 1 in Exhibit 20-1.

Exhibit 20-1
Analysis Cases for AWSC Intersections



If traffic is present on the other approaches, as well as on the subject approach, the saturation headway (the time between subsequent vehicle departures) on the subject approach will increase somewhat, depending on the degree of conflict that results between the subject approach vehicles and the vehicles on the other approaches. In Case 2, some uncertainty is introduced with a vehicle on the opposing approach, and thus the saturation headway will be greater than for Case 1. In Case 3, vehicles on one of the conflicting approaches further restrict the departure rate of vehicles on the subject approach, and the saturation headway will be longer than for Case 1 or Case 2. In Case 4, two vehicles are waiting on opposing or conflicting approaches, and saturation headways are even longer. When vehicles are present on all approaches, as in Case 5, saturation headways are the longest of any of the cases because the potential for conflict between vehicles is greatest. The increasing degree of potential conflict translates directly into longer driver decision times and longer saturation headways. Because no traffic signal controls the stream movement or allocates the right-of-way to each conflicting traffic stream, the rate of departure is controlled by the interactions between the traffic streams.

Capacity of an AWSC can be described by saturation headway, departure headway, and service time.

Therefore, the operation at an AWSC intersection can be described numerically by a few key time-based terms:

- The saturation headway, h_{si}, is the time between departures of successive vehicles on a given approach for a particular case (case i), as described above, assuming a continuous queue.
- The departure headway, h_d, is the average time between departures of successive vehicles on a given approach accounting for the probability of each possible case.
- The service time, t_s, is the average time spent by a vehicle in first position waiting to depart. It is equal to the departure headway minus the time it takes a vehicle to move from second position into first position (the move-up time, m).

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The intersection analysis boundaries for an AWSC analysis are assumed to be those of an isolated intersection; that is, no upstream or downstream effects are accounted for in the analysis. The present methodology is limited to motor vehicles.

LEVEL-OF-SERVICE CRITERIA

The level-of-service (LOS) criteria for AWSC intersections are given in Exhibit 20-2. As the exhibit notes, LOS F is assigned if the volume-to-capacity (v/c) ratio of a lane exceeds 1.0, regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

Exhibit 20-2
LOS Criteria: Automobile 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: * For approaches and intersectionwide assessment, LOS is defined solely by control delay.

REQUIRED INPUT DATA

Analysis of an AWSC intersection requires the following data:

1. Number and configuration of lanes on each approach;
2. Percentage of heavy vehicles;
3. Turning movement demand flow rate for each entering lane or, alternatively, hourly demand volume and peak hour factor; and
4. Length of analysis period—generally a peak 15-min period within the peak hour, although any 15-min period can be analyzed.

SCOPE OF THE METHODOLOGY

This chapter focuses on the operation of AWSC intersections. This version of the AWSC intersection analysis procedures is primarily a result of studies conducted by National Cooperative Highway Research Program Project 3-46 (*1*).

LIMITATIONS OF THE METHODOLOGY

Automobile Mode

The methodologies in this chapter apply to isolated AWSC intersections with up to three lanes on each approach. They do not account for interaction effects with other intersections. The methodologies do not apply to AWSC intersections with more than four approaches. In addition, the effect of conflicting pedestrians on automobiles is not considered in this procedure. Conflicting pedestrian movements are likely to increase the saturation headway of affected vehicular movements, but the magnitude of this effect is unknown as of the publication of this edition of the HCM.

Pedestrian and Bicycle Modes

The current methodologies for analyzing LOS and delay at AWSC intersections do not extend to pedestrians and apply to bicycles only in limited situations that are not supported by research at the time of publication of this edition. As such, there are no set LOS standards that apply to pedestrians or bicycles at AWSC intersections, nor can pedestrian or bicycle delay, capacity, or quality of service be quantitatively assessed by using the procedures described in this chapter. Additional research on pedestrian and bicyclist behavior and operations at AWSC intersections needs to be done before procedures can be developed that adequately address these issues. A discussion of qualitative effects is included in the methodology section of this chapter.

2. METHODOLOGY

OVERVIEW

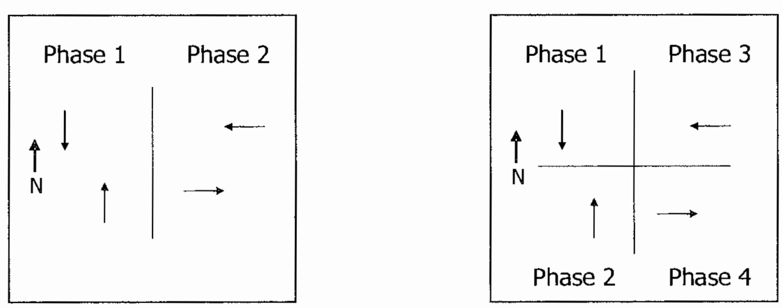
AWSC intersections require drivers on all approaches to stop before proceeding into the intersection. While giving priority to the driver on the right is a recognized rule in some areas, it is not a good descriptor of actual intersection operations. What happens is the development of a consensus of right-of-way that alternates between the drivers on the intersection approaches, a consensus that depends primarily on the intersection geometry and the arrival patterns at the stop line.

The methodology analyzes each intersection approach independently. The approach under study is called the subject approach. The opposing approach and the conflicting approaches create conflicts with vehicles on the subject approach.

Phase Patterns

A two-phase pattern, as shown in Exhibit 20-3(a), is observed at a standard four-leg AWSC intersection (one approach lane on each leg), where drivers from opposing approaches enter the intersection at roughly the same time. Some interruption of this pattern occurs when there are conflicts between certain turning maneuvers (such as a northbound left-turning vehicle and a southbound through vehicle), but generally the north–south streams alternate right-of-way with the east–west streams. A four-phase pattern, as shown in Exhibit 20-3(b), emerges at multilane four-leg intersections, where development of the right-of-way consensus is more difficult. Here drivers from each approach enter the intersection together as right-of-way passes from one approach to the next and each is served in turn. A similar three-phase pattern emerges at multilane three-leg intersections.

Exhibit 20-3
Operation Patterns at AWSC Intersections



(a) Two-phase (single-lane approaches) (b) Four-phase (multilane approaches)

Two cases for departure headways.

The headways of vehicles departing from the subject approach fall into one of two cases. If there are no vehicles on any of the other approaches, subject approach vehicles can enter the intersection immediately after stopping. However, if vehicles are waiting on a conflicting approach, a vehicle from the subject approach must wait for consensus with the next conflicting vehicle. The headways between consecutively departing subject approach vehicles will be shorter in the first case than in the second case. Thus, the headway for a departing subject approach vehicle depends on the degree of conflict experienced with vehicles on the other intersection approaches. The degree of conflict increases with two factors: the number of vehicles on the other approaches and the complexity of the intersection geometry.

Vehicle type and turning movement affect departure headway. These effects are captured empirically in the method.

Two other factors affect the departure headway of a subject approach vehicle: vehicle type and turning movement. The headway for a heavy vehicle will be longer than that for a passenger car. Furthermore, the headway for a left-turning vehicle will be longer than that for a through vehicle, which in turn will be longer than that for a right-turning vehicle.

In summary:

1. Standard four-leg AWSC intersections operate in either two-phase or four-phase patterns, based primarily on the complexity of the intersection geometry. Flows are determined by a consensus of right-of-way that alternates between the north–south and east–west streams (for a single-lane approach) or proceeds in turn to each intersection approach (for a multilane approach).
2. The headways between consecutively departing subject approach vehicles depend on the degree of conflict between these vehicles and the vehicles on the other intersection approaches. The degree of conflict is a function of the number of vehicles faced by the subject approach vehicle and of the number of lanes on the intersection approaches.
3. The headway of a subject approach vehicle also depends on its vehicle type and its turning maneuver (if any).

Capacity Concepts

Capacity defined.

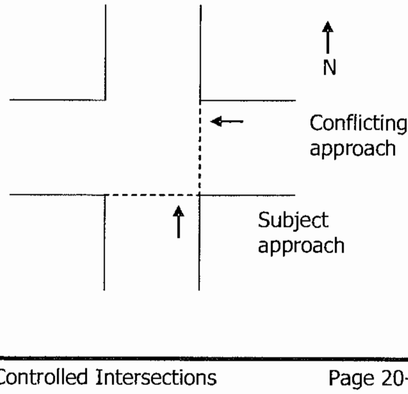
Capacity is defined as the maximum throughput on an approach given the flow rates on the other intersection approaches. The capacity model described here is an expansion of earlier work (2). The model is described for four increasingly complex cases: the intersection of two one-way streets with no turning movements, the intersection of two two-way streets with no turning movements, a generalized model for single-lane sites, and a generalized model for multilane sites. The methodology described later in this chapter is an implementation of the latter and most general case.

Intersection of Two One-Way Streets

The impact of turning movements is considered later, as part of the generalized models.

The first formulation of the model is based on the intersection of two one-way streets, each STOP-controlled. In this basic model, vehicles on either approach travel only straight through the intersection, as shown in Exhibit 20-4.

Exhibit 20-4
AWSC Configuration: Formulation 1



The saturation headway for a vehicle assumes one of two values: h_{s1} is the saturation headway if no vehicle is waiting on the conflicting approach, and h_{s2} is the saturation headway if the conflicting approach is occupied. The departure headway for vehicles on an approach is the expected value of this bivalued distribution. For the northbound approach, the mean service time is computed by Equation 20-1:

Equation 20-1

$$h_{d,N} = h_{s1}(1 - x_W) + h_{s2}x_W$$

where x_W is the degree of utilization of the westbound approach and is equal to the probability of finding at least one vehicle on that approach. Thus $1 - x_W$ is the probability of finding no vehicle on the westbound approach.

By symmetry, the mean service time for the westbound approach is given by Equation 20-2.

Equation 20-2

$$h_{d,W} = h_{s1}(1 - x_N) + h_{s2}x_N$$

Since the degree of utilization x is the product of the arrival rate λ and the mean departure headway h_d, the departure headways for each approach can be expressed in terms of the bivalued saturation headways and the arrival rates on each approach, as in Equation 20-3 and Equation 20-4.

Equation 20-3

$$h_{d,N} = \frac{h_{s1}[1 + \lambda_W(h_{s2} - h_{s1})]}{1 - \lambda_N\lambda_W(h_{s2} - h_{s1})^2}$$

Equation 20-4

$$h_{d,W} = \frac{h_{s1}[1 + \lambda_N(h_{s2} - h_{s1})]}{1 - \lambda_N\lambda_W(h_{s2} - h_{s1})^2}$$

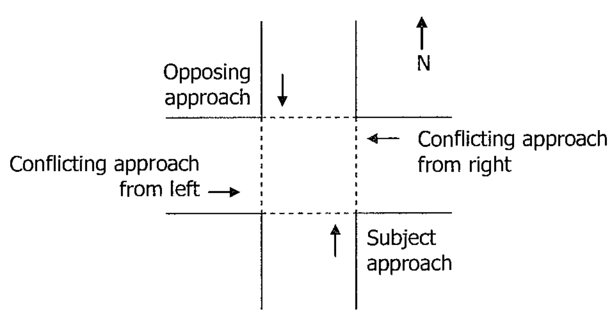
Intersection of Two Two-Way Streets

In this simplified model, the saturation headway for a vehicle assumes one of two values, h_{s1} or h_{s2}, because vehicles are again assumed to pass straight through the intersection. The departure headway for vehicles on an approach is the expected value of this bivalued distribution. A northbound vehicle will have a saturation headway of h_{s1} if the eastbound and westbound approaches are empty simultaneously. The probability of this event is the product of the probability of an empty westbound approach and the probability of an empty eastbound approach. The departure headway for the northbound vehicle is computed with Equation 20-5. See Exhibit 20-5.

Equation 20-5

$$h_{d,N} = h_{s1}(1 - x_E)(1 - x_W) + h_{s2}[1 - (1 - x_E)(1 - x_W)]$$

Exhibit 20-5
AWSC Configuration: Formulation 2



Unlike Formulation 1, it is not possible to solve directly for the departure headway in terms of a combination of arrival rates and the bivalued saturation headways. The departure headway on any approach depends on, or is directly coupled with, the traffic intensity on the two conflicting approaches. This coupling prevents a direct solution. However, it is possible to solve for the departure headway on each approach in an iterative manner, by using a system of equations similar in form to Equation 20-5.

Generalized Model for Single-Lane Sites

Capacity is determined by an iterative procedure.

The generalized model is based on five saturation headway values, each reflecting a different level or degree of conflict faced by the subject approach driver. Exhibit 20-6 specifies the conditions for each case and the probability of occurrence of each. The probability of occurrence is based on the degree of utilization on the opposing and conflicting approaches. The essence of the model, and its complexity, is evident when one realizes that the traffic intensity on one approach is computed from its capacity, which in turn depends on the traffic intensity on the other approaches. The interdependence of the traffic flow on all intersection approaches creates the need for iterative calculations to obtain stable estimates of departure headway and service time—and thus capacity.

Exhibit 20-6
Probability of Degree-of-Conflict Case

Degree-of-Conflict Case	Approach: Opp	Approach: Con-L	Approach: Con-R	Probability of Occurrence
1	N	N	N	$(1-x_O)(1-x_{CL})(1-x_{CR})$
2	Y	N	N	$(x_O)(1-x_{CL})(1-x_{CR})$
3	N	Y	N	$(1-x_O)(x_{CL})(1-x_{CR})$
3	N	N	Y	$(1-x_O)(1-x_{CL})(x_{CR})$
4	Y	N	Y	$(x_O)(1-x_{CL})(x_{CR})$
4	Y	Y	N	$(x_O)(x_{CL})(1-x_{CR})$
4	N	Y	Y	$(1-x_O)(x_{CL})(x_{CR})$
5	Y	Y	Y	$(x_O)(x_{CL})(x_{CR})$

Note: Opp = opposing approach, Con-L = conflicting approach from the left, Con-R = conflicting approach from the right, N = no, Y = yes.

The probability, $P(C_i)$, for each degree-of-conflict case given in Exhibit 20-6 can be computed with Equation 20-6 through Equation 20-10. The degrees of utilization on the opposing approach, the conflicting approach from the left, and the conflicting approach from the right are given by x_O, x_{CL}, and x_{CR}, respectively.

$$P(C_1)=(1-x_O)(1-x_{CL})(1-x_{CR})$$ **Equation 20-6**

$$P(C_2)=(x_O)(1-x_{CL})(1-x_{CR})$$ **Equation 20-7**

$$P(C_3)=(1-x_O)(x_{CL})(1-x_{CR})+(1-x_O)(1-x_{CL})(x_{CR})$$ **Equation 20-8**

$$P(C_4)=(x_O)(1-x_{CL})(x_{CR})+(x_O)(x_{CL})(1-x_{CR})+(1-x_O)(x_{CL})(x_{CR})$$ **Equation 20-9**

$$P(C_5)=(x_O)(x_{CL})(x_{CR})$$ **Equation 20-10**

The departure headway for an approach is the expected value of the saturation headway distribution, computed by Equation 20-11:

Equation 20-11

$$h_d = \sum_{i=1}^{5} P(C_i) h_{si}$$

where $P(C_i)$ is the probability of the degree-of-conflict case C_i and h_{si} is the saturation headway for that case, given the traffic stream and geometric conditions of the intersection approach.

Capacity is determined by increasing volume on the subject approach until x > 1.0.

The capacity is computed by incrementally increasing the volume on the subject approach until the degree of utilization exceeds 1.0. This flow rate is the maximum possible flow or throughput on the subject approach under the conditions used as input to the analysis.

Generalized Model for Multilane Sites

Saturation headways at multilane sites are typically longer than at single-lane sites, all other conditions being equal. This situation is the result of two factors:

- A larger intersection (i.e., greater number of lanes) requires more travel time through the intersection, thus increasing the saturation headway; and
- Additional lanes also result in an increasing degree of conflict with opposing and conflicting vehicles, again increasing driver decision time and the saturation headway.

By contrast, some movements may not conflict with each other as readily at multilane sites as at single-lane sites. For example, a northbound vehicle turning right may be able to depart simultaneously with an eastbound through movement if the two vehicles are able to occupy separate receiving lanes when departing to the east. Consequently, in some cases, the saturation headway may be lower at multilane sites.

The theory described earlier proposed that the saturation headway is a function of the directional movement of the vehicle, the vehicle type, and the degree of conflict faced by the subject vehicle. This theory is extended here for multilane sites with respect to the concept of degree of conflict: saturation headway is affected to a large extent by the number of opposing and conflicting vehicles faced by the subject driver. For example, in degree-of-conflict Case 2, a subject vehicle is faced only by a vehicle on the opposing approach. At a two-lane approach intersection, there can be either one or two vehicles on the opposing approach. Each degree-of-conflict case is expanded to consider the number of vehicles present on each of the opposing and conflicting approaches. The cases are defined in Exhibit 20-7 and Exhibit 20-8 for two-lane and three-lane approaches, respectively.

For multilane sites, separate saturation headway values are computed for the number of vehicles faced by the subject vehicle for each degree-of-conflict case. This calculation requires a further extension of the service time model to account for the increased number of subcases. These combinations can be further subdivided if a vehicle can be present on any lane on a given approach.

Exhibit 20-7
Degree-of-Conflict Cases for Two-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2
3		x		1, 2
			x	
4	x	x		2, 3, 4
	x		x	
		x	x	
5	x	x	x	3, 4, 5, 6

Exhibit 20-8
Degree-of-Conflict Cases for Three-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	Number of Opposing and Conflicting Vehicles
1				0
2	x			1, 2, 3
3		x		1, 2, 3
			x	
4	x	x		2, 3, 4, 5, 6
	x		x	
		x	x	
5	x	x	x	3, 4, 5, 6, 7, 8, 9

The probability of a vehicle being at the stop line in a given lane is x, the degree of utilization. The product of the six degrees of saturation, encompassing each of the six lanes on the opposing or conflicting approaches (two lanes on the opposing approach and two lanes on each of the conflicting approaches), gives the probability of any particular combination occurring.

The iterative procedure to compute the departure headways and capacities for each approach as a function of the departure headways on the other approaches is the same as described earlier. However, the additional subcases clearly increase the complexity of this computation.

AUTOMOBILE MODE

The AWSC intersection methodology for the automobile mode is applied through a series of steps that relate to input data, saturation headways, departure headways, service time, capacity, and LOS. They are illustrated in Exhibit 20-9.

Step 1: Convert Movement Demand Volumes to Flow Rates

Flow rates for each turning movement at the intersection must be converted from hourly volumes in vehicles per hour (veh/h) to peak 15-min flow rates in vehicles per hour as given in Equation 20-12:

$$v_i = \frac{V_i}{PHF}$$

Equation 20-12

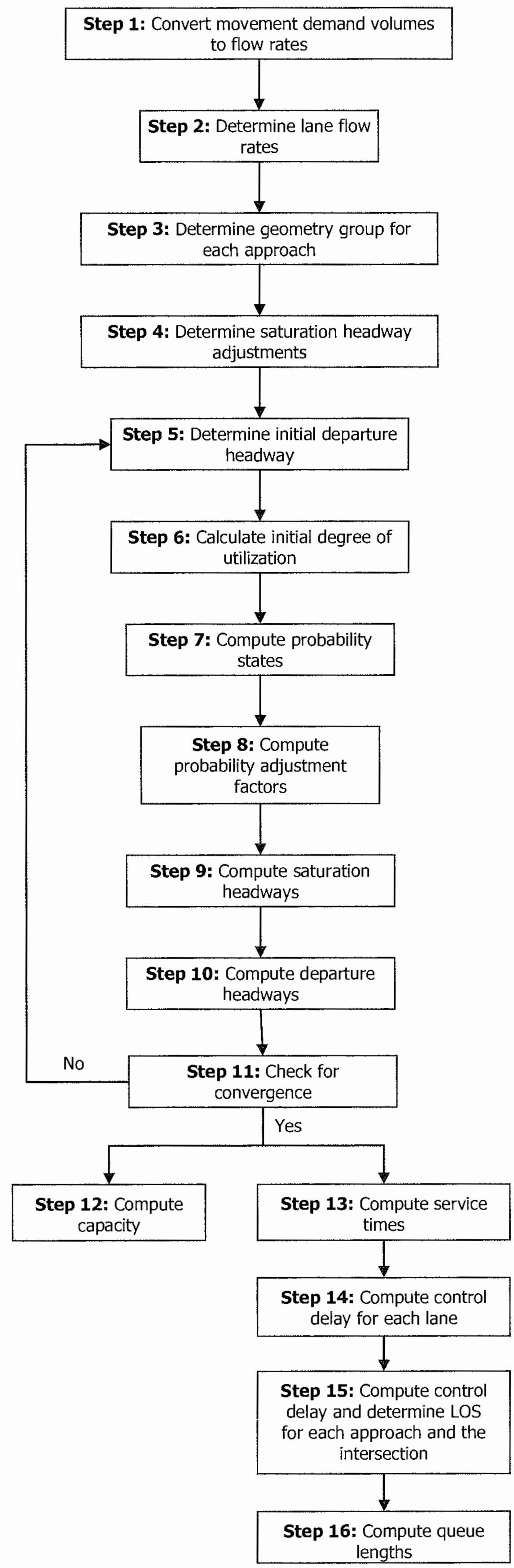
where

v_i = demand flow rate for movement i (veh/h),

V_i = demand volume for movement i (veh/h), and

PHF = peak hour factor.

Exhibit 20-9
AWSC Intersection Methodology



Step 2: Determine Lane Flow Rates

For multilane approaches, the flow rate for each lane by movement is determined. If a certain movement can use more than one lane and its traffic volume distribution per lane is unknown, an equal distribution of volume among the lanes can be assumed.

Step 3: Determine Geometry Group for Each Approach

Exhibit 20-10 is consulted to determine the geometry group for each approach. The geometry group is needed to look up base saturation headways and headway adjustment factors.

Exhibit 20-10
Geometry Groups

Intersection Configuration	Number of Lanes: Subject Approach	Number of Lanes: Opposing Approach	Number of Lanes: Conflicting Approaches[a]	Geometry Group
Four leg or T	1	0 or 1	1	1
Four leg or T	1	0 or 1	2	2
Four leg or T	1	2	1	3a/4a
T	1	2	2	3b
Four leg	1	2	2	4b
Four leg or T	1	0 or 1	3	5
	1	3	1	
	2	0, 1, or 2	1 or 2	
	3	0 or 1	1	
	3	0 or 1	2 or 3	
	3	2 or 3	1	
Four leg or T	1	3	2	6
	1	2	3	
	1	3	3	
	2	3	1, 2, or 3	
	2	0, 1, 2 or 3	3	
	3	2 or 3	2 or 3	

Note: [a] If the number of lanes on conflicting approaches is different, the higher of the two should be used.

Step 4: Determine Saturation Headway Adjustments

The headway adjustment for each lane is computed by Equation 20-13. Saturation headway adjustments for left turns, right turns, and heavy vehicles are given in Exhibit 20-11.

Equation 20-13

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

where

h_{adj} = headway adjustment (s),

$h_{LT,adj}$ = headway adjustment for left turns (see Exhibit 20-11) (s),

$h_{RT,adj}$ = headway adjustment for right turns (see Exhibit 20-11) (s),

$h_{HV,adj}$ = headway adjustment for heavy vehicles (see Exhibit 20-11) (s),

P_{LT} = proportion of left-turning vehicles in the lane,

P_{RT} = proportion of right-turning vehicles in the lane, and

P_{HV} = proportion of heavy vehicles in the lane.

Exhibit 20-11
Saturation Headway Adjustments by Geometry Group

Factor	Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
	Saturation Headway Adjustment (s)							
LT	0.2	0.2	0.2	0.2	0.2	0.2	0.5	0.5
RT	−0.6	−0.6	−0.6	−0.6	−0.6	−0.6	−0.7	−0.7
HV	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.7

Note: LT = left turns, RT = right turns, HV = heavy vehicles.

Step 5: Determine Initial Departure Headway

The process of determining departure headways (and thus service times) for each of the lanes on each of the approaches is iterative. For the first iteration, an initial departure headway of 3.2 s should be assumed. For subsequent iterations, the calculated values of departure headway from the previous iteration should be used if the calculation has not converged (see Step 11).

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 with Equation 20-14. If it is not the final iteration, and the degree of utilization exceeds 1, then the degree of utilization should be reset to 1.

Equation 20-14

$$x = \frac{vh_d}{3{,}600}$$

Step 7: Compute Probability States

The probability state of each combination i is found with Equation 20-15.

Equation 20-15

$$P(i) = \prod_j P(a_j)$$

where

j = O1 (opposing approach, Lane 1), O2 (opposing approach, Lane 2), CL1 (conflicting left approach, Lane 1), CL2 (conflicting left approach, Lane 2), CR1 (conflicting right approach, Lane 1), and CR2 (conflicting right approach, Lane 2) for a two-lane, two-way AWSC intersection;

$P(a_j)$ = probability of a_j, computed on the basis of Exhibit 20-12, where V_j is the lane flow rate; and

a_j = 1 (indicating a vehicle present) or 0 (indicating no vehicle present in the lane) (values of a_j for each lane in each combination i are listed in Exhibit 20-13).

Exhibit 20-12
Probability of a_j

a_j	V_j	$P(a_j)$
1	0	0
0	0	1
1	> 0	x_j
0	> 0	$1 - x_j$

Note: x is the degree of utilization defined in Equation 20-14.

Tables for three-lane approaches are given in Chapter 32, STOP-Controlled Intersections: Supplemental

Exhibit 20-13 provides the 64 possible combinations when alternative lane occupancies are considered for two-lane approaches. A 1 indicates that a vehicle is in the lane, and a 0 indicates that a vehicle is not in the lane. A similar table for three lanes on each approach is provided in Chapter 32 in Volume 4.

Exhibit 20-13
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Two-Lane Approaches, by Lane)

i	DOC Case	Number of Vehicles	Opposing Approach L1	Opposing Approach L2	Conflicting Left Approach L1	Conflicting Left Approach L2	Conflicting Right Approach L1	Conflicting Right Approach L2
1	1	0	0	0	0	0	0	0
2	2	1	1	0	0	0	0	0
3			0	1	0	0	0	0
4		2	1	1	0	0	0	0
5	3	1	0	0	1	0	0	0
6			0	0	0	1	0	0
7			0	0	0	0	1	0
8			0	0	0	0	0	1
9		2	0	0	1	1	0	0
10			0	0	0	0	1	1
11	4	2	0	0	0	1	0	1
12			0	0	1	0	0	1
13			0	0	1	0	1	0
14			0	0	0	1	1	0
15			0	1	0	1	0	0
16			1	0	1	0	0	0
17			0	1	0	0	1	0
18			1	0	0	1	0	0
19			0	1	1	0	0	0
20			0	1	0	0	0	1
21			1	0	0	0	1	0
22			1	0	0	0	0	1
23		3	0	0	0	1	1	1
24			0	0	1	1	0	1
25			0	0	1	1	1	0
26			1	0	1	1	0	0
27			1	1	1	0	0	0
28			1	1	0	0	1	0
29			1	1	0	0	0	1
30			0	1	1	1	0	0
31			1	0	0	0	1	1
32			0	0	1	0	1	1
33			1	1	0	1	0	0
34			0	1	0	0	1	1
35		4	1	1	0	0	1	1
36			0	0	1	1	1	1
37			1	1	1	1	0	0
38	5	3	0	1	0	1	0	1
39			1	0	0	1	1	0
40			0	1	1	0	1	0
41			0	1	0	1	1	0
42			0	1	1	0	0	1
43			1	0	1	0	0	1
44			1	0	0	1	0	1
45			1	0	1	0	1	0
46		4	1	0	0	1	1	1
47			0	1	1	1	1	0
48			0	1	1	1	0	1
49			1	0	1	0	1	1
50			1	0	1	1	1	0
51			0	1	0	1	1	1
52			1	1	1	0	0	1
53			1	0	1	1	0	1
54			0	1	1	0	1	1
55			1	1	0	1	1	0
56			1	1	0	1	0	1
57			1	1	1	0	1	0
58		5	1	0	1	1	1	1
59			1	1	0	1	1	1
60			1	1	1	0	1	1
61			0	1	1	1	1	1
62			1	1	1	1	1	0
63			1	1	1	1	0	1
64		6	1	1	1	1	1	1

Note: DOC case is the degree-of-conflict case, number of vehicles is the total number of vehicles on the opposing and conflicting approaches, L1 is Lane 1, and L2 is Lane 2.

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 20-16 through Equation 20-20 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined (assuming the 64 cases presented in Exhibit 20-13).

Equation 20-16

$$P(C_1) = P(1)$$

Equation 20-17

$$P(C_2) = \sum_{i=2}^{4} P(i)$$

Equation 20-18

$$P(C_3) = \sum_{i=5}^{10} P(i)$$

Equation 20-19

$$P(C_4) = \sum_{i=11}^{37} P(i)$$

Equation 20-20

$$P(C_5) = \sum_{i=38}^{64} P(i)$$

The probability adjustment factors are then computed with Equation 20-21 through Equation 20-25.

Equation 20-21

$$AdjP(1) = \alpha\left[P(C_2) + 2P(C_3) + 3P(C_4) + 4P(C_5)\right]/1$$

Equation 20-22

$$AdjP(2) \text{ through } AdjP(4) = \alpha\left[P(C_3) + 2P(C_4) + 3P(C_5) - P(C_2)\right]/3$$

Equation 20-23

$$AdjP(5) \text{ through } AdjP(10) = \alpha\left[P(C_4) + 2P(C_5) - 3P(C_3)\right]/6$$

Equation 20-24

$$AdjP(11) \text{ through } AdjP(37) = \alpha\left[P(C_5) - 6P(C_4)\right]/27$$

Equation 20-25

$$AdjP(38) \text{ through } AdjP(64) = -\alpha\left[10P(C_5)\right]/27$$

where α equals 0.01 (or 0.00 if correlation among saturation headways is not taken into account).

The adjusted probability $P'(i)$ for each combination is simply the sum of $P(i)$ and $AdjP(i)$, as given by Equation 20-26.

Equation 20-26

$$P'(i) = P(i) + AdjP(i)$$

Step 9: Compute Saturation Headways

The saturation headway h_{si} is the sum of the base saturation headway as presented in Exhibit 20-14 and the saturation headway adjustment factor from Step 4. It is shown in Equation 20-27.

Equation 20-27

$$h_{si} = h_{base} + h_{adj}$$

Case	No. of Veh.	Base Saturation Headway (s) Group 1	Group 2	Group 3a	Group 3b	Group 4a	Group 4b	Group 5	Group 6
1	0	3.9	3.9	4.0	4.3	4.0	4.5	4.5	4.5
2	1	4.7	4.7	4.8	5.1	4.8	5.3	5.0	6.0
	2							6.2	6.8
	≥3								7.4
3	1	5.8	5.8	5.9	6.2	5.9	6.4	6.4	6.6
	2							7.2	7.3
	≥3								7.8
4	2	7.0	7.0	7.1	7.4	7.1	7.6	7.6	8.1
	3							7.8	8.7
	4							9.0	9.6
	≥5								12.3
5	3	9.6	9.6	9.7	10.0	9.7	10.2	9.7	10.0
	4							9.7	11.1
	5							10.0	11.4
	≥6							11.5	13.3

Exhibit 20-14
Saturation Headway Values by Case and Geometry Group

Step 10: Compute Departure Headways

The departure headway of the approach is the expected value of the saturation headway distribution, given by Equation 20-28.

$$h_d = \sum_{i=1}^{64} P'(i) h_{si}$$

Equation 20-28

where i represents each combination of the five degree-of-conflict cases and h_{si} is the saturation headway for that combination.

Step 11: Check for Convergence

The calculated values of h_d are checked against the initial values assumed for h_d. If the values change by more than 0.1 s (or a more precise measure of convergence), Steps 5 through 10 are repeated until the values of departure headway for each lane do not change significantly.

Step 12: Compute Capacity

Capacity is estimated for a stated set of opposing and conflicting volumes.

The capacity of each approach is computed under the assumption that the flows on the opposing and conflicting approaches are constant. The given flow rate on the subject lane is increased and the departure headways are computed for each approach until the degree of utilization for the subject lane reaches 1. When this occurs, the final value of the subject approach flow rate is the maximum possible throughput or capacity of this lane.

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 with Equation 20-29.

$$t_s = h_d - m$$

Equation 20-29

where t_s is the service time, h_d is the departure headway, and m is the move-up time (2.0 s for Geometry Groups 1 through 4; 2.3 s for Geometry Groups 5 and 6).

Step 14: Compute Control Delay for Each Lane

The delay experienced by a motorist is made up of a number of factors that relate to control, geometrics, traffic, and incidents. Control delay is the difference between the travel time that is actually experienced and the reference travel time that would result during conditions in the absence of traffic control or conflicting traffic.

Equation 20-30 can be used to compute control delay for each lane.

Equation 20-30

$$d = t_s + 900T\left[(x-1)+\sqrt{(x-1)^2+\frac{h_d x}{450T}}\right]+5$$

where

d = average control delay (s/veh),

x = $vh_d/3{,}600$ = degree of utilization,

t_s = service time (s),

h_d = departure headway (s), and

T = length of analysis period (h).

Step 15: Compute Control Delay and Determine LOS for Each Approach and the Intersection

The control delay for an approach is calculated by computing a weighted average of the delay for each lane on the approach, weighted by the volume in each lane. The calculation is shown in Equation 20-31.

Equation 20-31

$$d_{\text{approach}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

d_{approach} = control delay for the approach (s/veh),

d_i = control delay for lane i (s/veh), and

v_i = flow rate for lane i (veh/h).

The control delay for the intersection as a whole is similarly calculated by computing a weighted average of the delay for each approach, weighted by the volume on each approach. It is shown in Equation 20-32.

Equation 20-32

$$d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

$d_{\text{intersection}}$ = control delay for the entire intersection (s/veh),

d_i = control delay for approach i (s/veh), and

v_i = flow rate for approach i (veh/h).

The LOS for each approach and for the intersection are determined with Exhibit 20-2 and the computed values of control delay.

Step 16: Compute Queue Lengths

Research (3) has determined that the methodology for predicting queues at TWSC intersections can be applied to AWSC intersections. As such, the mean queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest.

Equation 20-33 can be used to calculate the 95th percentile queue for each approach lane.

$$Q_{95} \approx \frac{900T}{h_d}\left[(x-1)+\sqrt{(x-1)^2+\frac{h_d x}{150T}}\right]$$

Equation 20-33

where

Q_{95} = 95th percentile queue (veh),

x = $vh_d/3{,}600$ = degree of utilization,

h_d = departure headway (s), and

T = length of analysis period (h).

PEDESTRIAN MODE

Data collection and research are needed to determine an appropriate LOS methodology for pedestrians at AWSC intersections.

Applying the LOS procedures used to determine pedestrian delay at TWSC intersections to AWSC intersections does not produce intuitive or usable results. The TWSC delay calculations apply only for crossings where conflicting traffic is not STOP-controlled (i.e., pedestrians crossing the major street at a TWSC intersection). Approaches where conflicting traffic is STOP-controlled (i.e., pedestrians crossing the minor street at a TWSC intersection) are assumed to result in negligible delay for pedestrians, as vehicles are required to stop and wait for conflicting vehicle and pedestrian traffic before proceeding.

As such, applying the TWSC methodology to pedestrians at AWSC intersections results in negligible delay for all pedestrians at all approaches. The reality of AWSC intersection operations for pedestrians is much different, however, and generally results in at least some delay for pedestrians. The amount of delay incurred will depend on a number of operating and geometric characteristics of the intersection in question. While no quantitative methodology accounting for these factors is available, several of the most important factors are discussed qualitatively below.

The operational characteristics of AWSC intersections for pedestrians largely depend on driver behavior. In most cases, drivers are legally required to yield to pedestrians crossing or preparing to cross AWSC intersections. However, it should be expected that operations differ significantly depending on enforcement levels, region of the country, and location (e.g., urban, suburban, or rural).

Traffic Volumes

At intersections with relatively low traffic volumes, there are generally no standing queues of vehicles at AWSC approaches. In these cases, pedestrians attempting to cross an approach of the intersection will typically experience little or no delay, as they will be able to proceed almost immediately after reaching the intersection.

At AWSC intersections with higher volumes, there are typically standing queues of motor vehicles on each approach. These intersections operate in a two-phase or four-phase sequence, as described earlier and depicted in Exhibit 20-3. In these situations, the arrival of a pedestrian does not typically disrupt the normal phase operations of the intersection. Rather, the pedestrian is often forced to wait until the phase arrives for vehicles in the approach moving adjacent to the pedestrian.

Under a scenario in which the intersection functions under the operations described above for pedestrians, average pedestrian delay might be expected to be half of the time needed to cycle through all phases for the particular intersection, assuming random arrival of pedestrians. However, several other factors may also affect pedestrian delay and operations at AWSC intersections, as described below.

Number of Approach Lanes

As the number of approach lanes at AWSC intersections increases, pedestrian crossing distance increases proportionally, resulting in significantly longer pedestrian crossing times compared with single-lane intersections. In addition, vehicles already in the intersection or about to enter the intersection take longer to complete their movement. As a result, pedestrians at multilane AWSC intersections may wait longer before taking their turn to cross.

Proportion of Turning Traffic

The ability of a pedestrian to cross at an AWSC intersection may also depend on the proportion of through motor vehicle traffic to turning motor vehicle traffic. As described above, pedestrians may often cross during the phase in which adjacent motor vehicle traffic traverses the intersection. However, when an adjacent motor vehicle is turning, that vehicle will conflict with pedestrians attempting to cross. Because of the additional conflicts with pedestrians created by turning vehicles at AWSC intersections, pedestrian delay may be expected to rise as the proportion of turning vehicles increases, similar to the effect that turning proportion has on vehicular delay.

Pedestrian Volumes

Under most circumstances, there is adequate capacity for all pedestrians queued for a given movement at an AWSC intersection to cross simultaneously with adjacent motor vehicle traffic. However, in locations with very high pedestrian volumes, this may not be the case. The total pedestrian capacity of a particular AWSC intersection phase is limited by both the width of the crosswalk (how many pedestrians can cross simultaneously) and driver behavior.

In situations in which not all queued pedestrians may cross during a particular phase, pedestrian delay will increase, as some pedestrians will be forced to wait through an additional cycle of intersection phases before crossing. However, pedestrian volumes in this range are unlikely to occur often; rather, intersections with pedestrian volumes high enough to cause significant delay are typically signalized.

BICYCLE MODE

Where bicycles queue with motor vehicles on AWSC approaches, the procedures described to estimate motor vehicle delay can be applied to bicycles. However, bicycles differ from motor vehicles in that they do not queue linearly at STOP signs. Instead, multiple bicycles often cross at the same time as the adjacent vehicular traffic stream. This phenomenon has not been researched as of the time of publication of this edition of the HCM and is not explicitly included in the methodology.

Where an AWSC approach provides a bicycle lane, bicycle delay will be significantly different and, in general, lower than motor vehicle delay. The exception is bicycles intending to turn left; those cyclists will typically queue with motor vehicles. Where bicycle lanes are available, bicycles are able to move unimpeded until reaching the stop line, as the bike lane allows the cyclist to pass any queued motor vehicles on the right. In this situation, bicycles will still incur delay upon reaching the intersection.

In most cases, bicycles will be able to travel through the intersection concurrently with adjacent motor vehicle traffic. This, in effect, results in multilane operations, with the bike lane serving as the curb lane, meaning that bicycles will be delayed from the time of arrival at the intersection until the adjacent motor vehicle phase occurs. As noted above, multiple bicycles will likely be able to cross simultaneously through the intersection. Finally, even where bicycle lanes are not available, many cyclists still pass queued motor vehicles on the right, resulting in lower effective bicycle delay compared with motor vehicle delay.

3. APPLICATIONS

DEFAULT VALUES

A comprehensive presentation of potential default values for interrupted-flow facilities is available elsewhere (*4*). These defaults cover the key characteristics of peak hour factor (*PHF*) and percent heavy vehicles (%HV). Recommendations are based on geographic region, population, and time of day. All general default values for interrupted flow facilities may be applied to analysis of AWSC intersections in the absence of field data or projections of conditions.

Both demand volumes and the number and configuration of lanes at the intersection are site specific and do not lend themselves to default values. The following default values may be applied to an AWSC intersection analysis:

- Peak hour factor (*PHF*) = 0.92
- Percent heavy vehicles (%HV) = 3%

As the number of default values used in any analysis increases, the accuracy of the result becomes more approximate and may differ significantly from the actual outcome, depending on local conditions.

ESTABLISH INTERSECTION ANALYSIS BOUNDARIES

This methodology assumes that the AWSC intersection under investigation is isolated. When interaction effects are likely between the subject AWSC intersection and other intersections (e.g., queue spillback, demand starvation), the use of alternative tools may result in a more accurate analysis. Analysis boundaries may also include different demand scenarios related to time of day or to different development scenarios that produce different demand flow rates.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design 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 (veh/h), heavy vehicle percentages for each approach, peak hour factor for all demand volumes, and lane configuration. The outputs of an operational analysis are estimates of capacity and control delay. The steps of the methodology, described in the Methodology section, are followed directly without modification.

Design Analysis

The operational analysis methodology for AWSC intersections can also be used for design analysis and planning and preliminary engineering 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 an AWSC 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 heavy-vehicle percentage and peak hour factor are typically estimated (or defaults used) when planning applications are performed.

USE OF ALTERNATIVE TOOLS

Except for the effects of interaction with other intersections, the limitations of the methodology that were stated earlier in this chapter have minimal potential to be addressed by alternative tools. Therefore, insufficient experience with alternative tools is available as of the time of publication of this edition of the HCM to support the development of useful guidance for their application to AWSC intersections.

4. EXAMPLE PROBLEM

An additional AWSC example problem is provided in Chapter 32, STOP-Controlled Intersections: Supplemental.

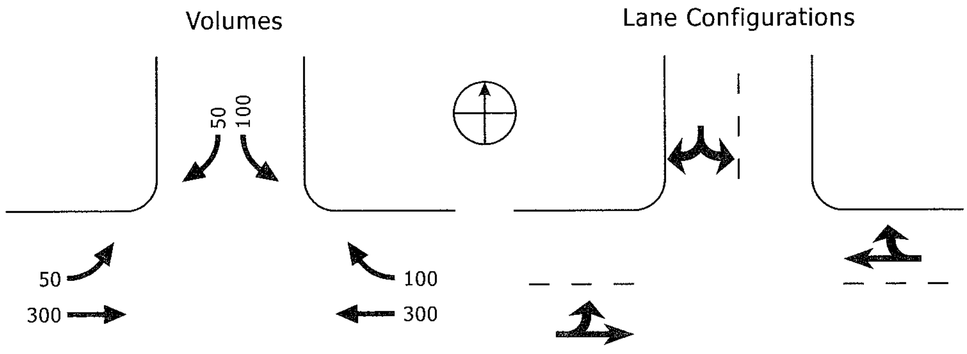
EXAMPLE PROBLEM 1: SINGLE-LANE, T-INTERSECTION

The Facts

The following describes this location's traffic and geometric characteristics:

- Three legs (T-intersection),
- One-lane entries on each leg,
- Percent heavy vehicles on all approaches = 2%,
- Peak hour factor = 0.95, and
- Volumes and lane configurations as shown below (Exhibit 20-15).

Exhibit 20-15
Volumes and Lane Configurations for Example Problem 1



Length of study period = 0.25 h

Comments

The use of a spreadsheet or software for AWSC intersection analysis is recommended because of the repetitive and iterative computations required.

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.

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 *PHF*. 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 20-10 shows that each approach should be assigned to Geometry Group 1.

Step 4: Determine Saturation Headway Adjustments

Exhibit 20-11 shows that 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 Geometry Group 1. Therefore, the saturation headway adjustment for the eastbound approach is as follows:

$$h_{adj} = h_{LT,adj}P_{LT} + h_{RT,adj}P_{RT} + h_{HV,adj}P_{HV}$$

$$h_{adj} = 0.2\frac{53}{53+316} - 0.6(0) + 1.7(0.02) = 0.063$$

Similarly, the saturation headway adjustment for the westbound approach is as follows:

$$h_{adj} = 0.2(0) - 0.6\frac{105}{105+316} + 1.7(0.02) = -0.116$$

Finally, the saturation headway adjustment for the southbound approach is as follows:

$$h_{adj} = 0.2\frac{105}{105+53} - 0.6\frac{53}{105+53} + 1.7(0.02) = -0.034$$

Steps 5 Through 11: Determine Departure Headway

These steps are iterative. The following narrative highlights some of the key calculations using the eastbound approach for Iteration 1 but does not attempt to reproduce all calculations for all iterations. Full documentation of the example problem is included in Chapter 32, STOP-Controlled Intersections: Supplemental.

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, for the eastbound approach is computed as follows:

$$x,EB = \frac{vh_d}{3{,}600} = \frac{(368)(3.2)}{3{,}600} = 0.327$$

$$x,WB = \frac{(421)(3.2)}{3{,}600} = 0.374$$

$$x,NB = \frac{vh_d}{3{,}600} = \frac{(158)(3.2)}{3{,}600} = 0.140$$

Step 7: Compute Probability States

The probability state of each combination i is determined with Equation 20-15.

$$P(i) = \prod_j P(a_j) = P(a_O)P(a_{CL})P(a_{CR})$$

For an intersection with single-lane approaches, only these eight cases from Exhibit 20-13 apply:

i	DOC Case	Number 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 (using Exhibit 20-6):

$$P(a_O) = 1 - x_O = 1 - 0.374 = 0.626 \quad \text{(no opposing present)}$$

$$P(a_{CL}) = 1 - x_{CL} = 1 - 0.140 = 0.860 \quad \text{(no conflicting from left present)}$$

$$P(a_{CR}) = 1 \quad \text{(no approach conflicting from right)}$$

Therefore,

$$P(1) = P(a_O)P(a_{CL})P(a_{CR}) = (0.626)(0.860)(1) = 0.538$$

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:

$$P(C_1) = P(1) = 0.538$$

$$P(C_2) = P(2) = 0.322$$

$$P(C_3) = P(5) + P(7) = 0.088 + 0 = 0.088$$

$$P(C_4) = P(13) + P(16) + P(21) = 0 + 0.052 + 0 = 0.052$$

$$P(C_5) = P(45) = 0$$

The probability adjustment factors for the nonzero cases are as follows:

$$AdjP(1) = 0.01[0.322 + 2(0.088) + 3(0.052) + 0]/1 = 0.0065$$

$$AdjP(2) = 0.01[(0.088) + 2(0.052) + 0 - 0.322]/3 = -0.0004$$

$$AdjP(5) = 0.01[(0.052) + 2(0) - 3(0.088)]/6 = -0.0004$$

$$AdjP(16) = 0.01[0 - 6(0.052)]/27 = -0.0001$$

Therefore, the adjusted probability for Combination 1, for example, is as follows:

$$P'(1) = 0.538 + 0.0065 = 0.5445$$

Step 9: Compute Saturation Headways

The base saturation headways for each combination can be determined with Exhibit 20-14. They are adjusted by using the adjustment factors calculated in Step 4 and added to the base saturation headways to determine saturation headways as follows (eastbound illustrated):

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 approach is the sum of the products of the adjusted probabilities and the saturation headways as follows (eastbound illustrated):

$$h_d = (0.5445)(3.963) + (0.3213)(4.763) + (0.0875)(5.863) + (0.0524)(7.063) = 4.57$$

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.

	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 Capacity

The capacity of each 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 approach capacity is approximately 720 veh/h, which is lower than the value that could be estimated by dividing the approach 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 other approaches, which in turn increases the departure headway of 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 20-29. For the eastbound approach (using a value for m of 2.0 for Geometry Group 1), the calculation is as follows:

$$t_s = h_d - m = 4.97 - 2.0 = 2.97$$

Step 14: Compute Control Delay

The control delay for each approach is computed with Equation 20-30 as follows (eastbound approach illustrated):

$$d = 2.97 + 900(0.25)\left[(0.508-1) + \sqrt{(0.508-1)^2 + \frac{4.97(0.508)}{450(0.25)}}\right] + 5 = 13.0 \text{ s}$$

By using Exhibit 20-2, the eastbound approach is assigned LOS B. A similar calculation for the westbound and southbound approaches yields 13.5 and 10.6 s, respectively.

The control delays for the approaches can be combined into an intersection control delay by using a weighted average as follows:

$$d = \frac{(13.0)(368) + (13.5)(421) + (10.6)(158)}{368 + 421 + 158} = 12.8 \text{ s}$$

This value of delay is assigned LOS B.

Step 15: Compute Queue Length

The 95th percentile queue for each lane is computed with Equation 20-33 as follows (eastbound approach illustrated):

$$Q_{95} \approx \frac{900(0.25)}{4.97}\left[(0.508-1) + \sqrt{(0.508-1)^2 + \frac{4.97(0.508)}{150(0.25)}}\right] = 2.9 \text{ veh}$$

This queue length would be reported as 3 vehicles.

Discussion

The results indicate that the intersection operates well with low delays.

These references are available in the Technical Reference Library in Volume 4.

5. REFERENCES

1. Kyte, M., Z. Tian, Z. Mir, Z. Hameedmansoor, W. Kittelson, M. Vandehey, B. Robinson, W. Brilon, L. Bondzio, N. Wu, and R. Troutbeck. *NCHRP Web Document 6: Capacity and Level of Service at Unsignalized Intersections: Final Report, Vol. 2—All-Way Stop-Controlled Intersections.* Transportation Research Board, National Research Council, Washington, D.C., April 1996.
2. Richardson, A. J. A Delay Model for Multiway Stop-Sign Intersections. In *Transportation Research Record 1112,* Transportation Research Board, National Research Council, Washington, D.C., 1987, pp. 107–114.
3. Tian, Z., and M. Kyte. Queue Length Models for All-Way Stop-Controlled Intersections. In *Transportation Research Record: Journal of the Transportation Research Board, No. 1988,* Transportation Research Board of the National Academies, Washington, D.C., 2006, pp. 63–66.
4. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses.* Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 21
ROUNDABOUTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Roundabouts are intersections with a generally circular shape, characterized by yield on entry and circulation around a central island (counterclockwise in the United States). Roundabouts have been used successfully throughout the world and are being used increasingly in the United States, especially since 1990.

Chapter 21, Roundabouts, presents concepts and procedures for analyzing these intersections. National Cooperative Highway Research Program Project 3-65 (*1*) provided a comprehensive database of roundabout operations for U.S. conditions on the basis of a study of 31 sites. The procedures that follow are largely founded on that study's recommendations. These procedures allow the analyst to assess the operational performance of an existing or planned one-lane or two-lane roundabout given traffic demand levels.

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The analytical procedure presented in this chapter assumes that the analysis boundaries are the roundabout itself, including associated pedestrian crosswalks. Alternative tools discussed in this chapter can, in some cases, expand the analysis boundaries to include adjacent intersections. The methodology presented here includes discussion of motor vehicles, pedestrians, and bicycles.

LEVEL OF SERVICE CRITERIA

The level of service (LOS) criteria for automobiles in roundabouts are given in Exhibit 21-1. As the table notes, LOS F is assigned if the volume-to-capacity ratio of a lane exceeds 1.0 regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

The thresholds in Exhibit 21-1 are based on the considered judgment of the Transportation Research Board Committee on Highway Capacity and Quality of Service. As discussed later in this chapter, roundabouts share the same basic control delay formulation with two-way and all-way STOP-controlled intersections, adjusting for the effect of YIELD control. However, at the time of publication of this edition of the *Highway Capacity Manual* (HCM), no research was available on traveler perception of quality of service at roundabouts. In the absence of such research, the service measure and thresholds have been made consistent with those for other unsignalized intersections, primarily on the basis of this similar control delay formulation.

Exhibit 21-1
LOS Criteria: Automobile Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a]	
	$v/c \leq 1.0$	$v/c > 1.0$
0–10	A	F
>10–15	B	F
>15–25	C	F
>25–35	D	F
>35–50	E	F
>50	F	F

Note: [a] For approaches and intersectionwide assessment, LOS is defined solely by control delay.

REQUIRED INPUT DATA

The following data are required to analyze a roundabout:

1. Number and configuration of lanes on each approach;
2. Either of the following:
   a. Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak 15 min, or
   b. Demand volume for each entering vehicular movement and each pedestrian crossing movement during the peak hour, and a peak hour factor for the hour;
3. Percentage of heavy vehicles;
4. Volume distribution across lanes for multilane entries; and
5. Length of analysis period, generally a peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

SCOPE OF THE METHODOLOGY

The methodology presented in this chapter focuses on the operation of roundabouts. The methodology does not account for the effects of adjacent traffic control devices such as nearby traffic signals or signalized pedestrian crossings. This version of the roundabout analysis procedures results primarily from studies conducted by National Cooperative Highway Research Program Project 3-65 (*1*). The chapter also includes a discussion of alternative tools that can model situations beyond the scope of the analytical methodology presented in this chapter.

The methodology does not necessarily apply to other types of circular intersections such as rotaries, neighborhood traffic circles, or signalized traffic circles, because these types of circular intersections usually have geometric or traffic control elements that deviate from those used in roundabouts. As a result, their operational performance may be significantly different from that experienced at roundabouts and thus cannot be accurately modeled by using the procedures in this chapter. More detail on the differentiation between roundabouts and other circular intersections can be found elsewhere (*2*, *3*).

LIMITATIONS OF THE METHODOLOGY

This chapter's methodology applies to isolated roundabouts with up to two entry lanes and up to one bypass lane per approach.

While the database on which these procedures are based is the most comprehensive developed for U.S. conditions, it does not cover all situations that may be encountered in practice. The chapter's methodology applies to isolated roundabouts with up to two entry lanes and up to one bypass lane per approach.

Automobile Mode

The methodology presented for automobiles covers typical roundabout facilities quite well, but it lacks examples of situations in which

- Upstream or downstream signals (including, but not limited to, pedestrian signals) significantly influence the performance of the roundabout;

- Priority reversal occurs, such as unusual forced entry conditions under extremely high flows;
- A high level of pedestrian or bicycle activity is present;
- The roundabout is in close proximity to one or more other roundabouts;
- More than two entry lanes are present on one or more approaches; or
- One or more entry lanes are of limited, or short, length (a flared design).

Priority reversal can occur when entering traffic dominates an entry, causing circulating traffic to yield.

A typical flared entry is one that widens from one approach lane to two entry lanes. Other flaring combinations, including flares of lane width, are possible.

Pedestrian Mode

Research on the operational performance of pedestrians at roundabouts is limited, in terms of the effect of both motor vehicles on pedestrians and pedestrians on motor vehicles. This chapter's methodologies include international models and analytical tools that have not been validated by research in the United States at the time of publication of this edition of the HCM. Additional research on pedestrian operations at roundabouts is needed to develop and refine procedures that adequately address these issues.

Bicycle Mode

Current methodologies to analyze LOS and delay at roundabouts only apply to bicycles in limited situations and have not been validated by research in the United States at the time of publication of this edition of the HCM. Additional research on bicycle behavior and operations at roundabouts is needed to develop procedures that adequately address these issues.

2. METHODOLOGY

OVERVIEW

This chapter presents procedures for analyzing roundabouts, introduces the unique characteristics of roundabout capacity, and presents terminology specific to roundabouts. For ease of reference, the following terms are defined:

v_e = entry flow rate,

v_c = conflicting flow rate, and

v_{ex} = exit flow rate.

Intersection analysis models generally fall into two categories. *Regression models* use field data to develop statistically derived relationships between geometric features and performance measures such as capacity and delay. *Analytical models* are based on traffic flow theory combined with the use of field measures of driver behavior, resulting in an analytic formulation of the relationship between those field measures and performance measures such as capacity and delay.

The procedure in this chapter uses a combination of regression and analytical models.

Both of these types of models are applicable to roundabouts. Gap-acceptance models are an example of an analytical model and are commonly applied for analyzing unsignalized intersections because they capture driver behavior characteristics directly and can be made site-specific by custom-tuning the values used for those parameters. However, simple gap-acceptance models may not capture all of the observed behavior, and more complex gap-acceptance models that account for limited priority or reverse priority are difficult to calibrate. Regression models are often used in these cases in which an understanding of driver behavior characteristics is incomplete. On the basis of recent analysis of U.S. field data, the procedure presented in this chapter incorporates a combination of simple, lane-based regression and gap-acceptance models for both single-lane and double-lane roundabouts.

CAPACITY CONCEPTS

The capacity of a roundabout approach is directly influenced by flow patterns. The three flows of interest, the entering flow, the circulating flow, and the exiting flow, are shown in Exhibit 21-2.

The capacity of a roundabout approach decreases as the circulating flow increases.

The capacity of an approach decreases as the conflicting flow increases. In general, the primary conflicting flow is the circulating flow that passes directly in front of the subject entry. While the circulating flow directly conflicts with the entry flow, the exiting flow may also affect a driver's decision to enter the roundabout. This phenomenon is similar to the effect of the right-turning stream approaching from the left side of a two-way STOP-controlled intersection. Until these drivers complete their exit maneuver or right turn, there may be some uncertainty in the mind of the driver at the yield or stop line about the intentions of the exiting or turning vehicle. However, even though it may have an influence in some cases, including this effect did not significantly improve the overall fit of the capacity models to the data (*1*) and therefore is not included in this chapter's models.

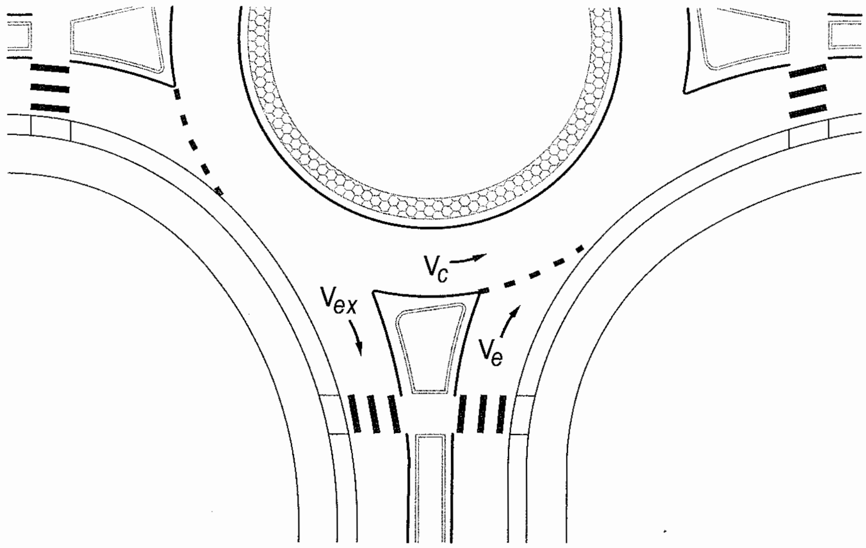


Exhibit 21-2
Analysis on One Roundabout Leg

When the conflicting flow rate approaches zero, the maximum entry flow is given by 3,600 s/h divided by the follow-up headway, which is analogous to the saturation flow rate for a movement receiving a green indication at a signalized intersection. At high levels of both entering and conflicting flow, limited priority (in which circulating traffic adjusts its headways to allow entering vehicles to enter), priority reversal (in which entering traffic forces circulating traffic to yield), and other behaviors may occur. In these cases, more complex analytical models or regression models, such as those incorporated into some of the alternative tools discussed later in this chapter, may give more realistic results.

When an approach operates over capacity during the analysis period, a condition known as *capacity constraint* may occur. During this condition, the actual circulating flow downstream of the constrained entry will be less than demand. The reduction in actual circulating flow may therefore increase the capacity of the affected downstream entries during this condition.

In addition, it has been suggested that origin–destination patterns have an influence on the capacity of a given entry (*4*, *5*). This effect was not identified in a recent study (*1*) and has not been incorporated into this chapter's models.

Both roundabout design practices and the public's use of roundabouts are still maturing in the United States. Many of the sites that formed the database for this chapter were less than 5 years old when the data were collected. Although the available data were insufficient to definitively answer the question of whether capacity increases with driver familiarity, anecdotal observations suggest that this may well be the case. At this early stage of their introduction to roundabouts, American drivers seem to be displaying more hesitation and caution in the use of roundabouts than their international counterparts, which in turn has resulted in a lower observed capacity than might be ultimately achievable. It is therefore likely that capacity (and volumes) will increase in the years to come as more roundabouts are constructed in the United States and as user familiarity grows. Such an increase in capacity over time would be consistent with the historically observed trends in capacity for freeway facilities and signalized intersections, for example. On the other hand, capacities in the

U.S. drivers presently seem to display more hesitation and caution in using roundabouts than drivers in other countries, which results in lower observed capacities. It is likely that capacities will increase in the future as U.S. drivers become more familiar with roundabouts.

United States over time may still be fundamentally different from those observed in other countries due to a variety of factors. These include limited use of turn indicators at roundabout exits by American drivers, differences in vehicle types, and the effect that the common use of STOP-controlled intersections (versus YIELD-controlled intersections) has had on drivers in the United States.

Single-Lane Roundabouts

The capacity of a single entry lane conflicted by one circulating lane (e.g., a single-lane roundabout, illustrated in Exhibit 21-3) is based on the conflicting flow. The equation for estimating the capacity is given as Equation 21-1:

Equation 21-1

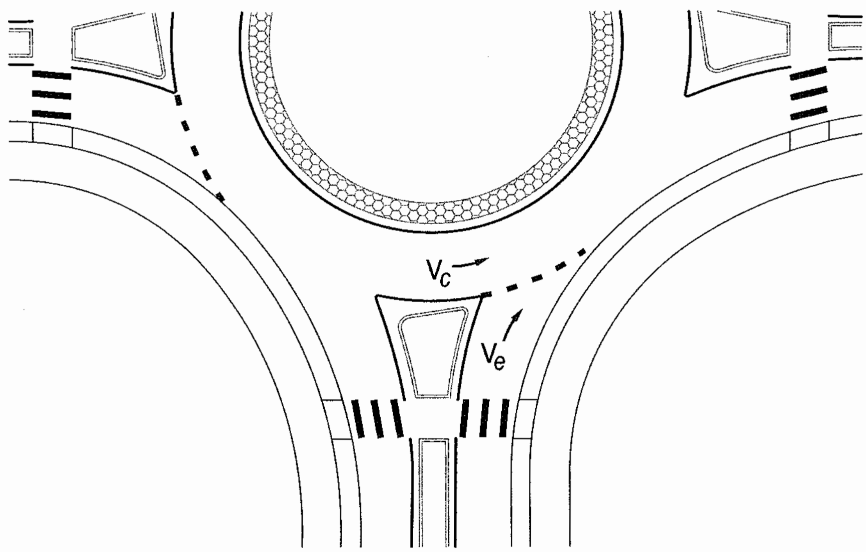
$$c_{e,pce} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce}}$$

where

$c_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h), and

$v_{c,pce}$ = conflicting flow rate (pc/h).

Exhibit 21-3
Example of One-Lane Entry Conflicted by One Circulating Lane



The capacity model given above reflects observations made at U.S. roundabouts in 2003. As noted previously, it is probable that U.S. roundabout capacity will increase to some degree with increased driver familiarity. In addition, communities with higher densities of roundabouts or generally more aggressive drivers may experience higher capacities. Therefore, local calibration of the capacity models is recommended to best reflect local driver behavior. This topic is discussed later in this chapter.

Multilane Roundabouts

Multilane roundabouts have more than one lane on at least one entry and at least part of the circulatory roadway. The number of entry, circulating, and exiting lanes may vary throughout the roundabout. Because of the many possible variations, the computational complexity is higher than for single-lane roundabouts.

The definition of headways and gaps for multilane facilities is more complicated than for single-lane facilities. If the circulating roadway truly functions as a multilane facility, then motorists at the entry perceive gaps in both the inside and outside lanes in some integrated fashion. Some drivers who choose to enter the roundabout via the right entry lane will yield to all traffic in the circulatory roadway due to their uncertainty about the path of the circulating vehicles. This uncertainty is more pronounced at roundabouts than at other unsignalized intersections due to the curvature of the circulatory roadway. However, some drivers in the right entry lane will enter next to a vehicle circulating in the inside lane if the circulating vehicle is not perceived to conflict. In addition, the behavior of circulating vehicles may be affected by the presence or absence of lane markings within the circulatory roadway. As a result, the gap-acceptance behavior of the right entry lane, in particular, is imperfect and difficult to quantify with a simple gap-acceptance model. This leads to an inclination toward using a regression-based model that implicitly accounts for these factors. More detail on the nuances of geometric design, pavement markings, and their relationship with operational performance can be found elsewhere (*2*, *3*).

For roundabouts with up to two circulating lanes, which is the only type of multilane roundabout addressed by the analytical methodology in this chapter, the entries and exits can be either one or two lanes wide (plus a possible right-turn bypass lane). The capacity model given above reflects observations made at a limited number of U.S. roundabouts in 2003. As discussed previously with single-lane roundabouts, local calibration of the capacity models (presented later in this chapter) is recommended to best reflect local driver behavior.

Capacity for Two-Lane Entries Conflicted by One Circulating Lane

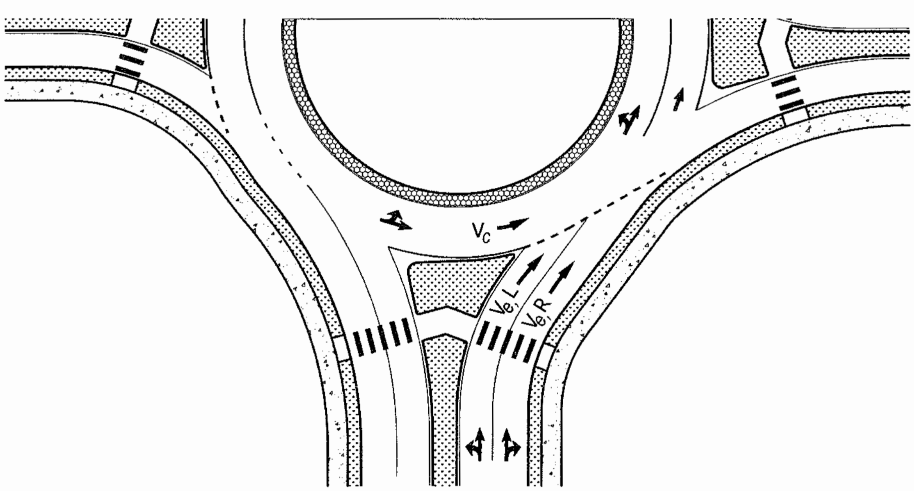
Equation 21-2 gives the capacity of each entry lane conflicted by one circulating lane (illustrated in Exhibit 21-4) as follows:

Equation 21-2

$$c_{e,pce} = 1{,}130 e^{(-1.0\times10^{-3}) v_{c,pce}}$$

where all variables are as defined previously.

Exhibit 21-4
Example of Two-Lane Entry Conflicted by One Circulating Lane



Capacity for One-Lane Entries Conflicted by Two Circulating Lanes

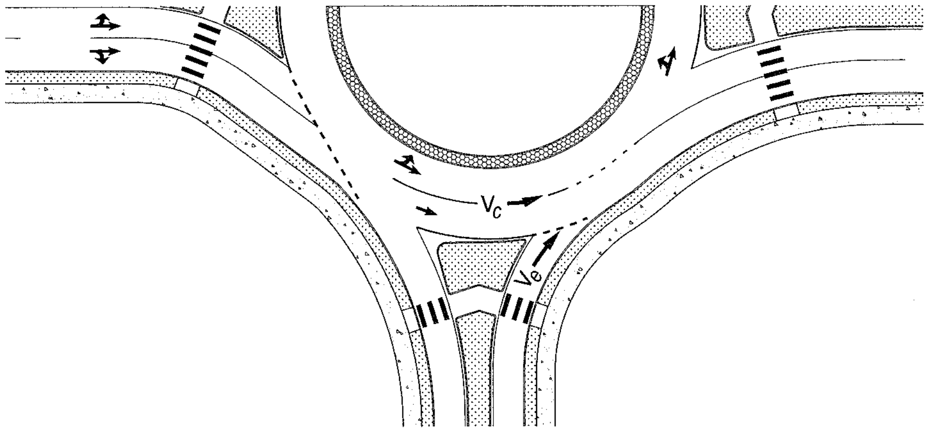
Equation 21-3 gives the capacity of a one-lane roundabout entry conflicted by two circulating lanes (illustrated in Exhibit 21-5) as follows:

Equation 21-3

$$c_{e,pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}}$$

where all variables are as defined previously ($v_{c,pce}$ is the total of both lanes).

Exhibit 21-5
Example of One-Lane Entry Conflicted by Two Circulating Lanes



Capacity for Two-Lane Entries Conflicted by Two Circulating Lanes

Equation 21-4 and Equation 21-5 give the capacity of the right and left lanes, respectively, of a two-lane roundabout entry conflicted by two circulating lanes (illustrated in Exhibit 21-6):

Equation 21-4

$$c_{e,R,pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}}$$

Equation 21-5

$$c_{e,L,pce} = 1{,}130e^{(-0.75\times10^{-3})v_{c,pce}}$$

where

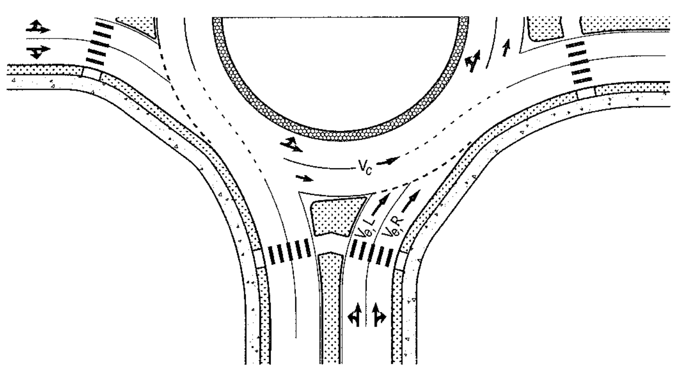
The capacity of the left lane of a roundabout approach is lower than the capacity of the right lane.

$c_{e,R,pce}$ = capacity of the right entry lane, adjusted for heavy vehicles (pc/h),

$c_{e,L,pce}$ = capacity of the left entry lane, adjusted for heavy vehicles (pc/h), and

$v_{c,pce}$ = conflicting flow rate (total of both lanes) (pc/h).

Exhibit 21-6
Example of Two-Lane Entry Conflicted by Two Circulating Lanes

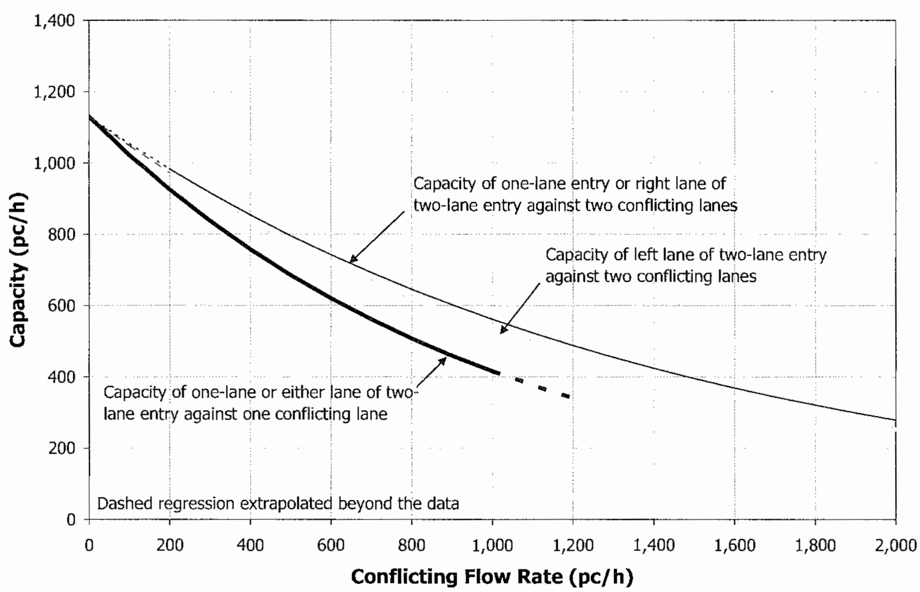


Field data (*1*) have found that drivers in the left lane have longer critical headways than drivers in the right lane. As a result, the capacity of the left lane is lower. Note that this research was able to observe sustained-queue conditions for only the right lane; Equation 21-4 represents a regression best fit that is also consistent with observed critical headways. The left-lane capacity given in Equation 21-5 is based on observed critical headways under both queued and nonqueued conditions.

The calculated capacities for each lane in passenger car equivalents per hour will be adjusted back to vehicles per hour, as described later in this section.

Exhibit 21-7 presents a plot showing Equation 21-1, Equation 21-4, and Equation 21-5. The dashed lines represent portions of the curves that lie outside the range of observed field data.

Exhibit 21-7
Capacity of Single-Lane and Multilane Entries



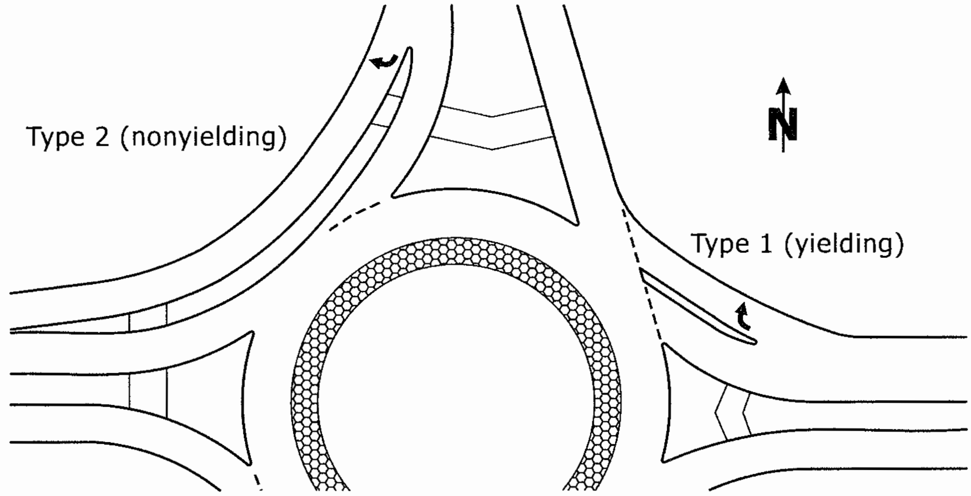
Right-Turn Bypass Lanes

Two common types of right-turn bypass lanes are used at both single-lane and multilane roundabouts. These are illustrated in Exhibit 21-8.

The bypass lane capacity procedure does not include the effect of drivers yielding to pedestrians.

The following sections describe each type of bypass lane. Note that in the United States, drivers in both types of bypass lane would generally be required to yield to pedestrians crossing the bypass lane. The capacity effect of drivers yielding to pedestrians has not been included in this analysis procedure.

Exhibit 21-8
Right-Turn Bypass Lanes



Type 1 (Yielding Bypass Lane)

A Type 1 bypass lane terminates at a high angle, with right-turning traffic yielding to exiting traffic. Right-turn bypass lanes were not explicitly included in the recent national research. However, the capacity of a yield bypass lane may be approximated by using one of the capacity formulas given previously by treating the exiting flow from the roundabout as the circulatory flow and treating the flow in the right-turn bypass lane as the entry flow.

The capacity for a bypass lane opposed by one exiting lane can be approximated by using Equation 21-6:

Equation 21-6

$$c_{\text{bypass},pce} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{ex,pce}}$$

The capacity for a bypass lane opposed by two exiting lanes can be approximated by using Equation 21-7:

Equation 21-7

$$c_{\text{bypass},pce} = 1{,}130e^{\left(-0.7\times10^{-3}\right)v_{ex,pce}}$$

where

$c_{\text{bypass},pce}$ = capacity of the bypass lane, adjusted for heavy vehicles (pc/h); and

$v_{ex,pce}$ = conflicting exiting flow rate (pc/h).

Type 2 (Nonyielding Bypass Lane)

A Type 2 bypass lane merges at a low angle with exiting traffic or forms a new lane adjacent to exiting traffic. The capacity of a merging bypass lane has not been assessed in the United States. Its capacity is expected to be relatively high due to a merging operation between two traffic streams at similar speeds.

Exit Capacity

German research (*6*) has suggested that the capacity of an exit lane, accounting for pedestrian and bicycle traffic in a typical urban area, is in the range of 1,200 to 1,300 vehicles per hour (veh/h). A Federal Highway Administration document used this information to provide guidance that exit flows exceeding 1,200 veh/h may indicate the need for a double-lane exit (*2*).

However, the analyst is cautioned to also evaluate exit lane requirements on the basis of vehicular lane numbers and arrangements. For example, a double-lane exit might be required to receive two through lanes in order to provide basic lane continuity along a corridor, regardless of the volume at the exit. Further guidance can be found elsewhere (2).

AUTOMOBILE MODE

The capacity of a given approach is computed by using the process illustrated in Exhibit 21-9.

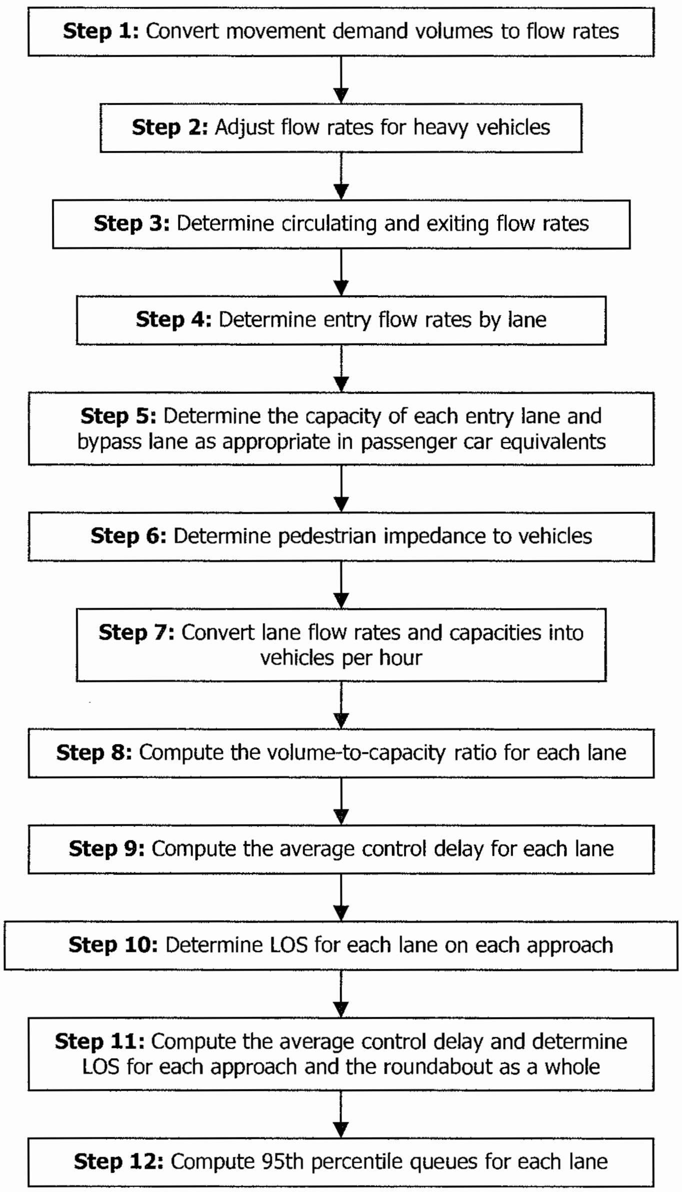


Exhibit 21-9
Roundabout Analysis Methodology

Step 1: Convert Movement Demand Volumes to Flow Rates

For an analysis of existing conditions in which 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 4.

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 21-8, through the use of a peak hour factor for the intersection:

Equation 21-8

$$v_i = \frac{V_i}{PHF}$$

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.

Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicle stream characteristics by using factors given in Exhibit 21-10.

Exhibit 21-10
Passenger Car Equivalencies

Vehicle Type	Passenger Car Equivalent, E_T
Passenger car	1.0
Heavy vehicle	2.0

The calculation to incorporate these values is given in Equation 21-9 and Equation 21-10:

Equation 21-9

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Equation 21-10

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

$v_{i,pce}$ = demand flow rate for movement i (pc/h),

v_i = demand flow rate for movement i (veh/h),

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of demand volume that consists of heavy vehicles, and

E_T = passenger car equivalent for heavy vehicles.

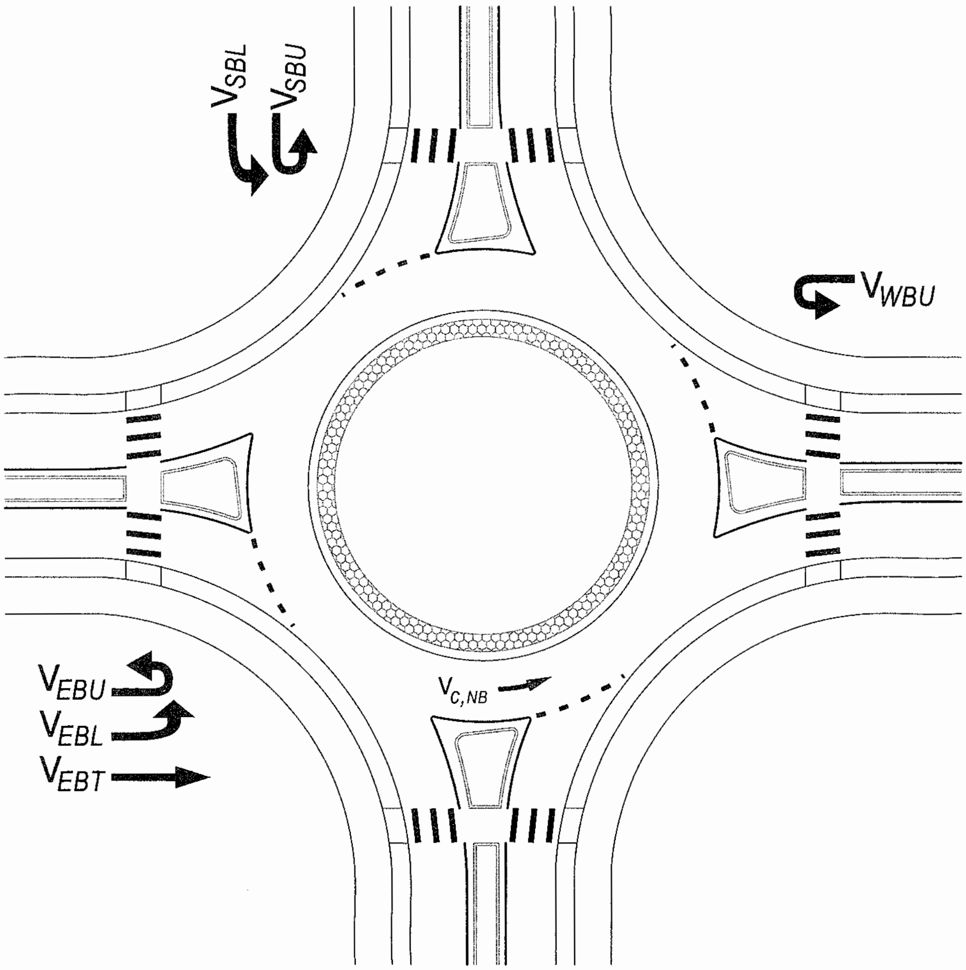
Step 3: Determine Circulating and Exiting Flow Rates

Circulating and exiting flow rates are calculated for each roundabout leg. Although the following sections present a numerical methodology for a four-leg roundabout, this methodology can be extended to any number of legs.

Circulating Flow Rate

The circulating flow opposing a given entry is defined as the flow conflicting with the entry flow (i.e., the flow passing in front of the splitter island next to the subject entry). The circulating flow rate calculation for the northbound circulating flow rate is illustrated in Exhibit 21-11 and numerically in Equation 21-11. All flows are in passenger car equivalents.

Exhibit 21-11
Calculation of Circulating Flow



Equation 21-11

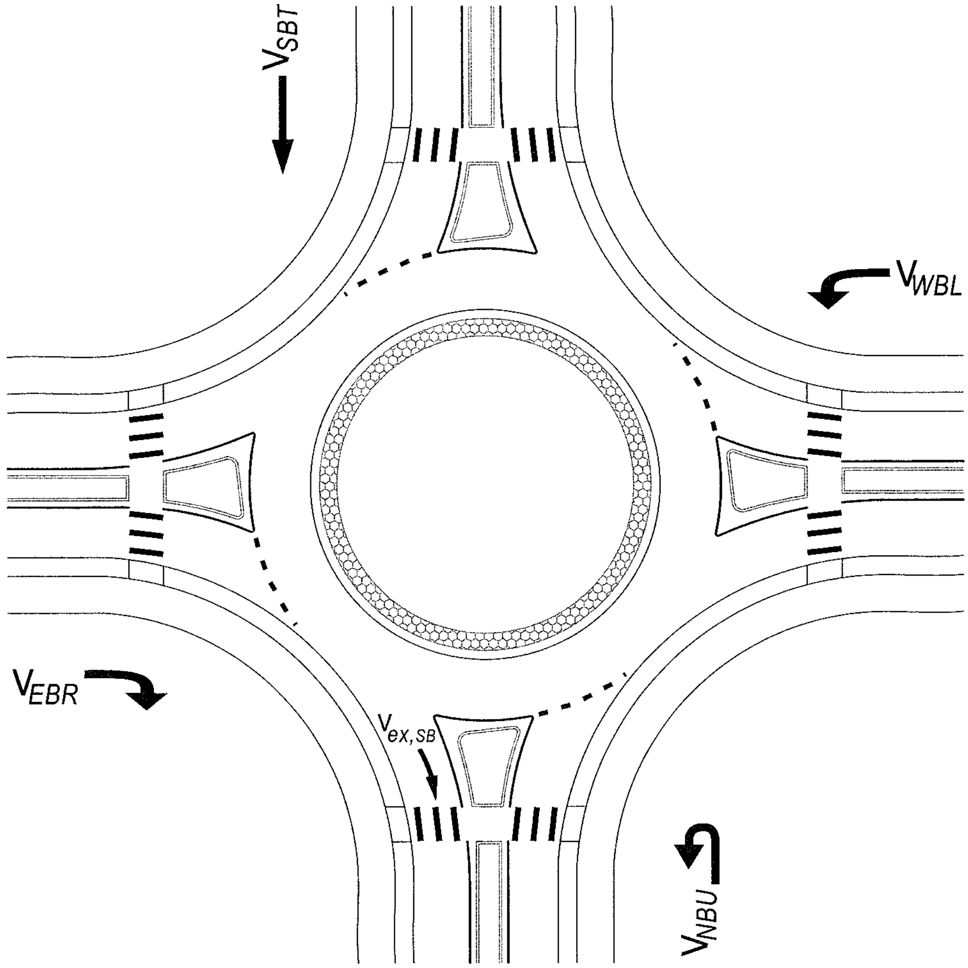
$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Exiting Flow Rate

The exiting flow rate for a given leg is used primarily in the calculation of conflicting flow for right-turn bypass lanes. The exiting flow calculation for the southbound exit is illustrated in Exhibit 21-12 and numerically in Equation 21-12. If a bypass lane is present on the immediate upstream entry, the right-turning flow using the bypass lane is deducted from the exiting flow. All flows are in passenger car equivalents.

If a bypass lane is present on the immediate upstream entry, the right-turning flow using the bypass lane is deducted from the exiting flow.

Exhibit 21-12
Calculation of Exiting Flow



Equation 21-12

$$v_{ex,pce} = v_{NBU,pce} + v_{WBL,pce} + v_{SBT,pce} + v_{EBR,pce} - v_{EBR,pce,bypass}$$

Step 4: Determine Entry Flow Rates by Lane

For single-lane entries, the entry flow rate is the sum of all movement flow rates using that entry. For multilane entries or entries with bypass lanes, or both, the following procedure may be used to assign flows to each lane:

1. If a right-turn bypass lane is provided, the flow using the bypass lane is removed from the calculation of the roundabout entry flows.
2. If only one lane is available for a given movement, the flow for that movement is assigned only to that lane.
3. The remaining flows are assumed to be distributed across all lanes, subject to the constraints imposed by any designated or de facto lane assignments and any observed or estimated lane utilization imbalances.

A de facto lane is one designated for multiple movements but that may operate as an exclusive lane due to a dominant movement demand. A common example is a left–through lane with a left-turn flow rate that greatly exceeds the through flow rate.

Five generalized multilane cases may be analyzed with this procedure. For cases in which a movement may use more than one lane, a check should first be made to determine what the assumed lane configuration may be. This may differ from the designated lane assignment based on the specific turning movement patterns being analyzed. These assumed lane assignments are given in Exhibit

21-13. For intersections with a different number of legs, the analyst should exercise reasonable judgment in assigning volumes to each lane.

Exhibit 21-13
Assumed (de facto) Lane Assignments

Designated Lane Assignment	Assumed Lane Assignment
LT, TR	If $v_U + v_L > v_T + v_{R,e}$: L, TR (de facto left-turn lane) If $v_{R,e} > v_U + v_L + v_T$: LT, R (de facto right-turn lane) Else LT, TR
L, LTR	If $v_T + v_{R,e} > v_U + v_L$: L, TR (de facto through–right lane) Else L, LTR
LTR, R	If $v_U + v_L + v_T > v_{R,e}$: LT, R (de facto left–through lane) Else LTR, R

Notes: v_U, v_L, v_T, and $v_{R,e}$ are the U-turn, left-turn, through, and nonbypass right-turn flow rates using a given entry, respectively.
L = left, LT = left–through, TR = through–right, LTR = left–through–right, and R = right.

On the basis of the assumed lane assignment for the entry and the lane utilization effect described above, flow rates can be assigned to each lane by using the formulas given in Exhibit 21-14. In this exhibit, $\%RL$ is the percentage of entry traffic using the right lane, $\%LL$ is the percentage of entry traffic using the left lane, and $\%LL + \%RL = 1$.

Exhibit 21-14
Volume Assignments for Two-Lane Entries

Case	Assumed Lane Assignment	Left Lane	Right Lane
1	L, TR	$v_U + v_L$	$v_T + v_{R,e}$
2	LT, R	$v_U + v_L + v_T$	$v_{R,e}$
3	LT, TR	$(\%LL)v_e$	$(\%RL)v_e$
4	L, LTR	$(\%LL)v_e$	$(\%RL)v_e$
5	LTR, R	$(\%LL)v_e$	$(\%RL)v_e$

Notes: v_U, v_L, v_T, and $v_{R,e}$ are the U-turn, left-turn, through, and nonbypass right-turn flow rates using a given entry, respectively.
L = left, LT = left–through, TR = through–right, LTR = left–through–right, and R = right.

Further discussion of lane use at multilane roundabouts, including conditions that may create unequal lane use, can be found in Chapter 33, Roundabouts: Supplemental, located in HCM Volume 4.

Step 5: Determine the Capacity of Each Entry Lane and Bypass Lane as Appropriate in Passenger Car Equivalents

The capacity of each entry lane and bypass lane is calculated by using the capacity equations discussed previously. Capacity equations for entry lanes are summarized in Exhibit 21-15; capacity equations for bypass lanes are summarized in Exhibit 21-16.

Exhibit 21-15
Capacity Equations for Entry Lanes

Entering Lanes	Conflicting Circulating Lanes	Capacity Equation
1	1	Equation 21-1
2	1	Each lane: Equation 21-2
1	2	Equation 21-3
2	2	Right lane: Equation 21-4; left lane: Equation 21-5

Exhibit 21-16
Capacity Equations for Bypass Lanes

Conflicting Exiting Lanes	Capacity Equation
1	Equation 21-6
2	Equation 21-7

Step 6: Determine Pedestrian Impedance to Vehicles

Pedestrian traffic can reduce the vehicular capacity of a roundabout entry if sufficient pedestrians are present and they assert the right-of-way typically granted pedestrians in most jurisdictions. Under high vehicular conflicting flows, pedestrians typically pass between queued vehicles on entry and thus have negligible additional impact on vehicular entry capacity. However, under low vehicular conflicting flows, pedestrians can effectively function as additional conflicting vehicles and thus reduce the vehicular capacity of the entry. The effect of pedestrians is more pronounced with increased pedestrian volume.

For one-lane roundabout entries, the model shown in Exhibit 21-17 can be used to approximate this effect (*6*). These equations are illustrated in Exhibit 21-18 and are based on the assumption that pedestrians have absolute priority.

Exhibit 21-17
Model of Entry Capacity Adjustment Factor for Pedestrians Crossing a One-Lane Entry (Assuming Pedestrian Priority)

Case	One-Lane Entry Capacity Adjustment Factor for Pedestrians
If $v_{c,pce} > 881$	$f_{ped} = 1$
Else if $n_{ped} \leq 101$	$f_{ped} = 1 - 0.000137 n_{ped}$
Else	$f_{ped} = \dfrac{1{,}119.5 - 0.715 v_{c,pce} - 0.644 n_{ped} + 0.00073 v_{c,pce} n_{ped}}{1{,}068.6 - 0.654 v_{c,pce}}$

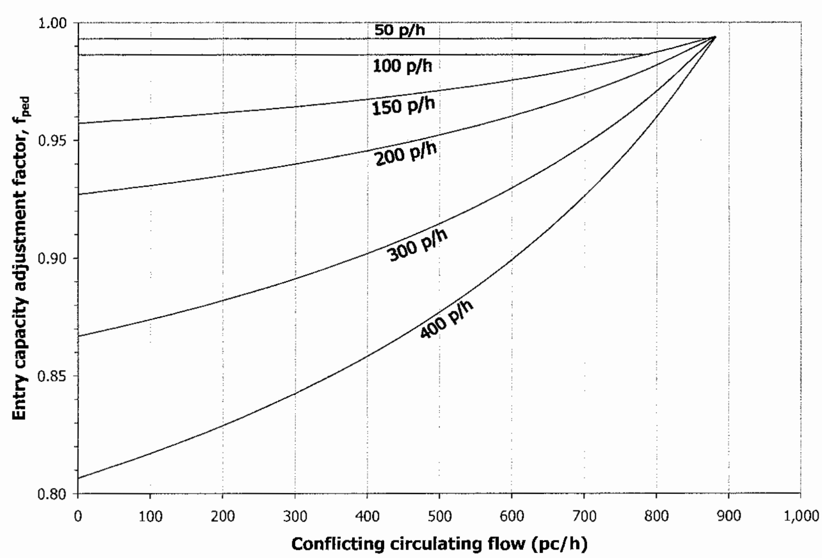
where

f_{ped} = entry capacity adjustment factor for pedestrians,

n_{ped} = number of conflicting pedestrians per hour (p/h), and

$v_{c,pce}$ = conflicting vehicular flow rate in the circulatory roadway, pc/h.

Exhibit 21-18
Illustration of Entry Capacity Adjustment Factor for Pedestrians Crossing a One-Lane Entry (Assuming Pedestrian Priority)



For two-lane entries, the model shown in Exhibit 21-19 can be used to approximate this effect (6). These equations are illustrated in Exhibit 21-20 and share the assumption as before that pedestrians have absolute priority.

Exhibit 21-19
Model of Entry Capacity Adjustment Factor for Pedestrians Crossing a Two-Lane Entry (Assuming Pedestrian Priority)

Case	Two-Lane Entry Capacity Adjustment Factor for Pedestrians
If $n_{ped} < 100$	$f_{ped} = \min\left[1 - \frac{n_{ped}}{100}\left(1 - \frac{1{,}260.6 - 0.329v_{c,pce} - 0.381 \times 100}{1{,}380 - 0.5v_{c,pce}}\right), 1\right]$
Else	$f_{ped} = \min\left[\frac{1{,}260.6 - 0.329v_{c,pce} - 0.381n_{ped}}{1{,}380 - 0.5v_{c,pce}}, 1\right]$

where

f_{ped} = entry capacity adjustment factor for pedestrians,

n_{ped} = number of conflicting pedestrians (p/h), and

$v_{c,pce}$ = conflicting vehicular flow rate in the circulatory roadway (pc/h).

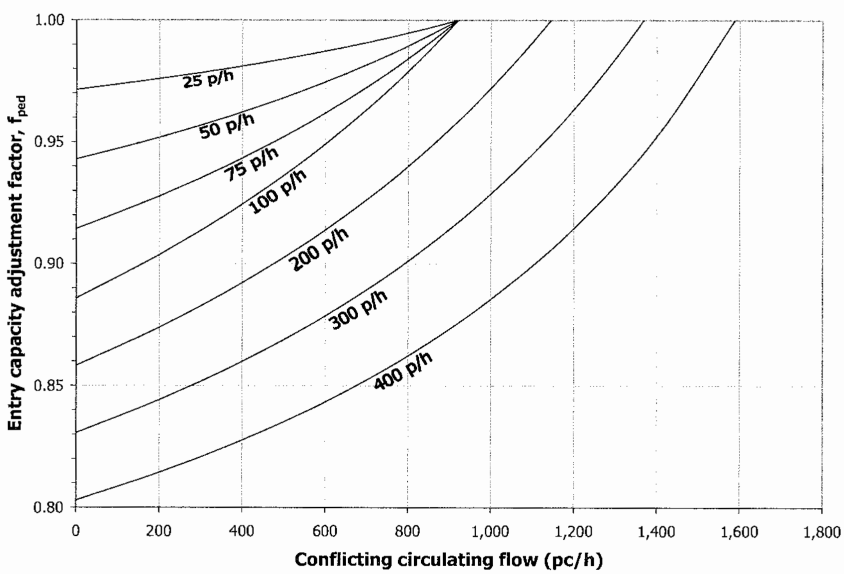


Exhibit 21-20
Illustration of Entry Capacity Adjustment Factor for Pedestrians Crossing a Two-Lane Entry (Assuming Pedestrian Priority)

Step 7: Convert Lane Flow Rates and Capacities into Vehicles per Hour

The flow rate for a given lane is converted back to vehicles per hour by multiplying the passenger-car-equivalent flow rate computed in the previous step by the heavy-vehicle factor for the lane as shown in Equation 21-13:

Equation 21-13

$$v_i = v_{i,PCE} f_{HV,e}$$

where

v_i = flow rate for lane i (veh/h),

$v_{i,PCE}$ = flow rate for lane i (pc/h), and

$f_{HV,e}$ = heavy-vehicle adjustment factor for the lane (see below).

Similarly, the capacity for a given lane is converted back to vehicles per hour as shown in Equation 21-14:

Equation 21-14

$$c_i = c_{i,PCE} f_{HV,e} f_{ped}$$

where

c_i = capacity for lane i (veh/h),

$c_{i,PCE}$ = capacity for lane i (pc/h),

$f_{HV,e}$ = heavy-vehicle adjustment factor for the lane (see below), and

f_{ped} = pedestrian impedance factor.

The heavy-vehicle adjustment factor for each entry lane can be approximated by taking a weighted average of the heavy-vehicle adjustment factors for each movement entering the roundabout (excluding a bypass lane if present) weighted by flow rate, as shown in Equation 21-15:

Equation 21-15

$$f_{HV,e} = \frac{f_{HV,U} v_{U,PCE} + f_{HV,L} v_{L,PCE} + f_{HV,T} v_{T,PCE} + f_{HV,R,e} v_{R,e,PCE}}{v_{U,PCE} + v_{L,PCE} + v_{T,PCE} + v_{R,e,PCE}}$$

where

$f_{HV,e}$ = heavy-vehicle adjustment factor for the entry lane,

$f_{HV,i}$ = heavy-vehicle adjustment factor for movement i, and

$v_{i,PCE}$ = demand flow rate for movement i (pc/h).

If specific lane-use assignment by heavy vehicles is known, heavy-vehicle adjustment factors can be calculated separately for each lane.

Pedestrian impedance is discussed later in this chapter.

Step 8: Compute the Volume-to-Capacity Ratio for Each Lane

For a given lane, the volume-to-capacity ratio x is calculated by dividing the lane's calculated capacity into its demand flow rate, as shown in Equation 21-16. Both input values are in vehicles per hour.

Equation 21-16

$$x_i = \frac{v_i}{c_i}$$

where

x_i = volume-to-capacity ratio of the subject lane i,

v_i = demand flow rate of the subject lane i (veh/h), and

c_i = capacity of the subject lane i (veh/h).

Step 9: Compute the Average Control Delay for Each Lane

Delay data collected for roundabouts in the United States suggest that control delays can be predicted in a manner generally similar to that used for other unsignalized intersections. Equation 21-17 shows the model that should be used to estimate average control delay for each lane of a roundabout approach:

$$d = \frac{3{,}600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{450T}}\right] + 5 \times \min[x,1]$$

Equation 21-17

The third term of this equation uses the calculated volume-to-capacity ratio or 1, whichever is less.

where

d = average control delay (s/veh),

x = volume-to-capacity ratio of the subject lane,

c = capacity of the subject lane (veh/h), and

T = time period (h) ($T = 0.25$ h for a 15-min analysis).

Equation 21-17 is the same as that for STOP-controlled intersections except that the "+ 5" term has been modified. This modification is necessary to account for the YIELD control on the subject entry, which does not require drivers to come to a complete stop when there is no conflicting traffic. At higher volume-to-capacity ratios, the likelihood of coming to a complete stop increases, thus causing behavior to resemble STOP control more closely.

Average control delay for a given lane is a function of the lane's capacity and degree of saturation. The analytical model used above to estimate average control delay assumes that there is no residual queue at the start of the analysis period. 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 calculated by the procedure may not be accurate due to the likely presence of a queue at the start of the time period. In addition, the conflicting demand for movements downstream of the movement operating over capacity may not be fully realized (in other words, the flow cannot get past the oversaturated entry and thus cannot conflict with a downstream entry). In these cases, an iterative approach that accounts for this effect and the carryover of queues from one time period to the next may be considered, as discussed elsewhere (*7*).

Step 10: Determine LOS for Each Lane on Each Approach

The LOS for each lane on each approach is determined by using Exhibit 21-1 and the computed or measured values of control delay.

Step 11: Compute the Average Control Delay and Determine LOS for Each Approach and the Roundabout as a Whole

The control delay for an approach is calculated by computing a weighted average of the delay for each lane on the approach, weighted by the volume in each lane. The calculation is shown in Equation 21-18. Note that the volume in the bypass lane should be included in the delay calculation for the approach. The LOS for each approach is determined by using Exhibit 21-1 and the computed or measured values of control delay.

Equation 21-18

$$d_{\text{approach}} = \frac{d_{LL}v_{LL} + d_{RL}v_{RL} + d_{\text{bypass}}v_{\text{bypass}}}{v_{LL} + v_{RL} + v_{\text{bypass}}}$$

The control delay for the intersection as a whole is similarly calculated by computing a weighted average of the delay for each approach, weighted by the volume on each approach. This is shown in Equation 21-19. The LOS for the intersection is determined by using Exhibit 21-1 and the computed or measured values of control delay.

Equation 21-19

$$d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}$$

where

$d_{\text{intersection}}$ = control delay for the entire intersection (s/veh),

d_i = control delay for approach i (s/veh), and

v_i = flow rate for approach i (veh/h).

Step 12: Compute 95th Percentile Queues for Each Lane

The 95th percentile queue for a given lane on an approach is calculated by using Equation 21-20:

Equation 21-20

$$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{150T}}\right]\left(\frac{c}{3{,}600}\right)$$

where

Q_{95} = 95th percentile queue (veh),

x = volume-to-capacity ratio of the subject lane,

c = capacity of the subject lane (veh/h), and

T = time period (h) ($T = 1$ for a 1-h analysis, $T = 0.25$ for a 15-min analysis).

The queue length calculated for each lane should be checked against the available storage. The queue in each lane may interact with adjacent lanes in one or more ways:

- If queues in adjacent lanes exceed the available storage, the queue in the subject lane may be longer than anticipated due to additional queuing from the adjacent lane.
- If queues in the subject lane exceed the available storage for adjacent lanes, the adjacent lane may be starved by the queue in the subject lane.

Should one or more of these conditions occur, a sensitivity analysis can be conducted with the methodology by varying the demand in each lane. The analyst may also use an alternative tool that is sensitive to lane-by-lane effects, as discussed in this chapter's Applications section.

PEDESTRIAN MODE

Limited research has been performed to date in the United States on the operational impacts of vehicular traffic on pedestrians at roundabouts. In the United States, pedestrians have the right-of-way either after entering a crosswalk or as they are about to enter the crosswalk, depending on specific state law. This is somewhat different from other countries that may establish absolute pedestrian right-of-way in some situations (typically urban) and absolute vehicular right-of-way in others (typically rural).

Much of the recent research focus on pedestrians in the United States has been in the area of assessing accessibility for pedestrians with vision disabilities. Research has found that some roundabouts present a challenge for blind and visually impaired pedestrians relative to sighted pedestrians, thus potentially bringing them out of compliance with the Americans with Disabilities Act (*8*). A variety of treatments has been or is being considered to improve roundabouts' accessibility to this group of pedestrians, including various types of signalization of pedestrian crossings. The analysis of these treatments can in some cases be performed by simple analytical methods presented in the HCM (e.g., the analysis procedure for the pedestrian mode in Chapter 19). However, in many cases, alternative tools will produce more accurate results. These are discussed later in this chapter.

Techniques to analyze the operational performance of pedestrians as provided in Chapter 19, Two-Way STOP-Controlled Intersections, can be applied with care at roundabouts. As noted in that chapter, vehicular yielding rates vary depending on crossing treatment, number of lanes, posted speed limit, and within individual sites (*9*). This makes modeling of pedestrian interactions imprecise. As a result, models to analyze vehicular effects on pedestrian travel should be applied with caution.

BICYCLE MODE

Use a passenger-car-equivalent factor of 0.5 for bicycles when treating them as motor vehicles.

As of the publication date of this edition of the HCM, no methodology specific to bicyclists has been developed to assess the performance of bicyclists at roundabouts, as few data are available in the United States to support model calibration. Depending on individual comfort level, ability, geometric conditions, and traffic conditions, bicyclists may either circulate as a motor vehicle or as a pedestrian. If bicyclists are circulating as motor vehicles, their effect can be approximated by combining bicyclist flow rates with other vehicles by using a passenger-car-equivalent factor of 0.5 (*2*). If bicyclists are circulating as pedestrians, their effect can be analyzed by using the methodology described previously for pedestrians. Further guidance on accommodating bicyclists at roundabouts can be found elsewhere (*2*).

3. APPLICATIONS

DEFAULT VALUES

No default values have been developed specifically for roundabouts. However, a comprehensive presentation of potential default values for interrupted-flow facilities is available (*10*), with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of peak hour factor and percent heavy vehicles. Recommendations are based on geographical region, population, and time of day. All general default values for interrupted-flow facilities may be applied to the analysis of roundabouts in the absence of field data or projections of conditions.

Demand volumes as well as the number and configuration of lanes at a roundabout are site-specific and thus do not lend themselves to default values. The following default values may be applied to a roundabout analysis:

- Peak hour factor = 0.92, and
- Percent heavy vehicles = 3%.

Default values for lane utilization on two-lane roundabout approaches are not provided in the above reference (*10*). In these cases, in the absence of field data, the effect of lane utilization imbalance can be approximated by using the assumed values given in Exhibit 21-21.

Exhibit 21-21
Assumed Default Values for Lane Utilization for Two-Lane Approaches

Lane Configuration	% Traffic in Left Lane[a]	% Traffic in Right Lane[a]
Left–through + through–right	0.47	0.53
Left–through–right + right	0.47	0.53
Left + left–through–right	0.53	0.47

Notes: [a] These values are generally consistent with observed values for through movements at signalized intersections. These values should be applied with care, particularly under conditions estimated to be near capacity.

Obviously, as the number of default values used in any analysis increases, the analysis result becomes more approximate and may be significantly different from the actual outcome, depending on local conditions.

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

Operational analysis takes traffic flow data and geometric configurations as input to determine operational performance.

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), heavy-vehicle percentages for each approach, peak hour factor for all hourly demand volumes (if not provided as 15-min volumes), and lane configuration. The outputs of an operational analysis will be estimates of capacity and control delay. The steps of the methodology, described in the Methodology section, are followed directly without modification.

Design Analysis

The operational analysis methodology described earlier in this chapter can be used for design purposes by using a given set of traffic flow data to determine iteratively the number and configuration of lanes that would be required to produce a given LOS.

Design analysis is used to determine the geometric configuration of a roundabout to produce a desired operational performance.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of a roundabout. 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 percent heavy vehicles and peak hour factor are typically estimated (or defaults used) when planning applications are performed.

Planning and preliminary engineering analysis is used to evaluate future conditions for which assumptions and estimates must be made.

CALIBRATION OF CAPACITY MODEL

The capacity models presented previously can be generalized by using the expressions in Equation 21-21 through Equation 21-23 as follows:

$$c_{pce} = Ae^{(-B\,v_c)}$$

Equation 21-21

$$A = \frac{3{,}600}{t_f}$$

Equation 21-22

$$B = \frac{t_c - (t_f/2)}{3{,}600}$$

Equation 21-23

where

c_{pce} = lane capacity, adjusted for heavy vehicles (pc/h),

v_c = conflicting flow (pc/h),

t_c = critical headway (s), and

t_f = follow-up headway (s).

Field measures of critical headway and follow-up headway can be used to calibrate the capacity models.

Therefore, the capacity model can be calibrated by using two parameters: the critical headway t_c and the follow-up headway t_f. An example illustrating this procedure is provided in Chapter 33, Roundabouts: Supplemental.

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 section contains specific guidance for the application of alternative tools to the analysis of roundabouts. The reader should also be familiar with the information and guidance on the design and evaluation of roundabouts (2, 3).

Two modeling approaches are used in the types of alternative tools commonly applied:

- *Deterministic intersection models.* These models represent vehicle flows as flow rates and are sensitive to various flow and geometric features of the roundabout, including lane numbers and arrangements or specific geometric dimensions (e.g., entry width, inscribed circle diameter), or both. The majority of these models are anchored to research conducted outside the United States (e.g., *11–14*). Some software implementations may include more than one model or employ extensions beyond the original fundamental research conducted within a particular country. Some deterministic models can model an entire network of intersections but generally assume no interaction effects between intersections, thus potentially limiting their application.
- *Stochastic network models.* These models represent vehicle flows by simulating individual vehicles and their car-following, lane-choice, and gap-acceptance decisions. The models are based on a variety of fundamental research studies on driver behavior (e.g., *15, 16*). By their nature, most stochastic models used for roundabouts can model an entire network of intersections, thus making them capable of modeling a broader range of problems. However, their data requirements are typically more intensive than for the deterministic intersection models. Most stochastic models are implemented in microsimulation tools.

Strengths of the HCM Procedure

The procedures in this chapter were based on extensive research supported by a significant quantity of field data. They have evolved over several years and represent a body of expert consensus. They produce unique deterministic results for a given set of inputs, and the capacity of each approach is an explicit part of the results. Alternative tools based on deterministic intersection models also produce a unique set of results, including capacities, for a given set of inputs, while those based on simulation may produce different results based on different random number sequences. Unique results from an analysis tool are important for some purposes such as development impact review.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

The procedures presented in this chapter cover many of the typical situations that a user may encounter in practice. However, there are sometimes applications for which alternative tools can produce a more accurate analysis. The following limitations, stated earlier in this chapter, may be addressed by using available simulation tools. The conditions beyond the scope of this chapter that are treated explicitly by alternative tools include

- Adjacent signals or roundabouts,
- Priority reversal under extremely high flows,
- High pedestrian or bicycle activity levels,
- More than two entry lanes on an approach, or

- Flared entry lanes.

A few of the more common applications of alternative tools to overcome the limitations of the procedures presented in this chapter will now be discussed.

Interaction Effects with Other Traffic Control Devices

Several common situations can be modeled with alternative tools:

- *Pedestrian signals or hybrid beacons at roundabout crosswalks.* These devices, described in detail in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (*17*), can be used in a variety of applications, including the following:
    - High vehicle flows in which naturally occurring gaps in vehicle traffic or vehicular yielding for pedestrians is insufficient;
    - High pedestrian flows in which unrestricted pedestrian crossing activity may create insufficient capacity for motor vehicles; and
    - Crossing situations in which pedestrians with vision or other impairments may not receive equivalent access to the crossing. This is a legal requirement in the United States under the Americans with Disabilities Act and is regulated by the U.S. Access Board (*8*).
- *Metering signals on roundabout approaches.* These signals are sometimes used in applications in which a dominant entering flow reduces downstream entry capacity to zero or nearly zero. A metering signal can create gaps in the dominant flow at regular intervals or as dictated by queuing at the downstream entry.
- *Signals used to give priority to other users.* These applications include at-grade rail crossings, emergency vehicle signals, and others.
- *Nearby intersections or traffic control devices at which queues or lane use effects interact.* These nearby intersections can have any type of control, including signalization, STOP control, or YIELD control (as at another roundabout). Applications could also include nonintersection treatments such as freeway ramp meters.

While some deterministic intersection tools can model these situations, they are often treated more satisfactorily by using stochastic network models.

Flared Entries or Short-Lane Applications

Flared entries or short-lane applications are sometimes used at roundabouts to add capacity at the entry without substantially widening the approach upstream of the entry. Common applications include flaring from one lane to two lanes at the entry or from two lanes to three lanes, although some international research has found capacity sensitivity to flaring in sub-lane-width increments (*13*).

The methodology presented in this chapter provides a mechanism for flagging conditions under which queues for a given lane may exceed available storage or block access to adjacent lanes. Alternative tools may provide more accurate modeling of these situations.

Three-Lane Roundabouts

Three-lane roundabouts are not included in the methodology described in this chapter but can be analyzed by a number of alternative tools. Note that no data for three-lane roundabouts are available in the source material (*1*) for this chapter's methodology, so the analyst should use care in estimating calibration parameters.

Adjustment of Simulation Parameters to the HCM Results

Calibration of any model used to analyze roundabouts is essential in producing realistic results that are consistent with field data. Ideally, field data should be used for calibration. For situations involving the assessment of hypothetical or proposed alternatives for which no field data exist, alternative tool results may be made more compatible with HCM results by adjusting alternative tool parameters to obtain a better match with the results obtained from the HCM procedures as follows:

- *Deterministic intersection models.* Typical calibration parameters for deterministic models include global adjustment factors that shift or shape the capacity model used by the model. These include adjustments to the intercept and slope of linear models or other shaping parameters of more complex analytical forms.
- *Stochastic network models.* Calibration of stochastic models is more challenging than for deterministic models because some calibration factors, such as factors related to driver aggressiveness, often apply globally to all elements of the network and not just to roundabouts. In other cases, the specific coding of the model can be fine-tuned to reflect localized driver behavior, including look-ahead points for gap acceptance and locations for discretionary and mandatory lane changes.

Step-by-Step Recommendations for Applying Alternative Tools

The following steps should be taken in applying an alternative tool in the analysis of roundabouts:

1. Identify the limitations of the HCM procedures that dictate the use of alternative tools.
2. Decide between a microscopic and a macroscopic modeling approach.
3. If possible, develop a simpler configuration that can be analyzed by the HCM procedures. Analyze the simple configuration by using both the HCM and the selected alternative tool. Make adjustments to the alternative tool parameters to obtain a better match with the HCM results.
4. Perform the analysis of the full configuration by using the alternative tool.
5. Interpret and present the results.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 29, Urban Street Facilities: Supplemental, includes an example of the application of a simulation tool to assess the effect of using a roundabout within a coordinated arterial signal system. The interactions between the roundabout and the arterial system are examined by using signal timing plans with different progression characteristics.

4. EXAMPLE PROBLEMS

Exhibit 21-22 List of Example Problems

Example Problem	Description	Application
1	Single-lane roundabout with bypass lanes	Operational analysis
2	Multilane roundabout	Operational analysis

EXAMPLE PROBLEM 1: SINGLE-LANE ROUNDABOUT WITH BYPASS LANES

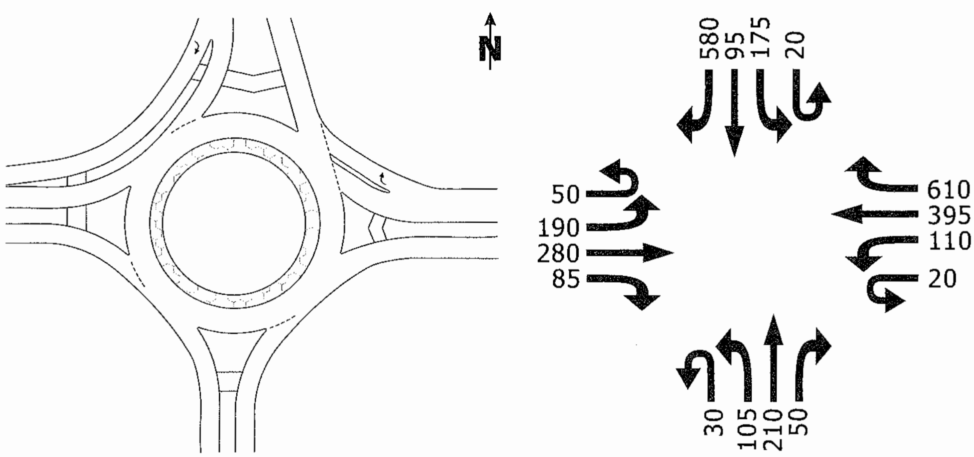
This is an example of an operational analysis. It uses traffic data and geometric characteristics to determine capacities, control delay, and LOS.

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Four legs,
- One-lane entries on each leg,
- A westbound right-turn bypass lane that yields to exiting vehicles,
- A southbound right-turn bypass lane that forms its own lane adjacent to exiting vehicles,
- Percent heavy vehicles for all movements = 2%,
- Peak hour factor = 0.94,
- Demand volumes and lane configurations as shown in Exhibit 21-23, and
- 50 p/h across the south leg and negligible pedestrian activity across the other three legs.

Exhibit 21-23 Demand Volumes and Lane Configurations for Example Problem 1



Comments

All input parameters are known, so no default values are needed or used.

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement volume given in the problem is converted to a demand flow rate by dividing by the peak hour factor. As an example, the northbound left-turn volume is converted to a flow rate as follows:

$$v_{NBL} = \frac{V_{NBL}}{PHF} = \frac{105}{0.94} = 112 \text{ pc/h}$$

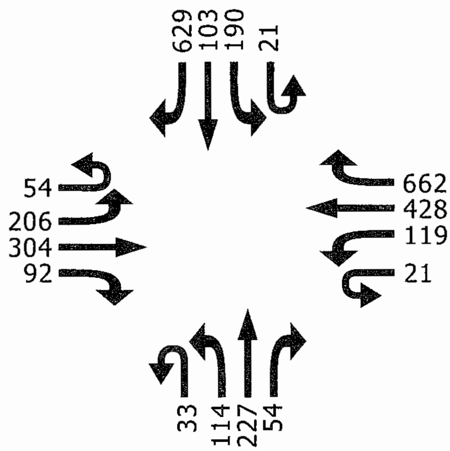
Step 2: Adjust Flow Rates for Heavy Vehicles

The flow rate for each movement may be adjusted to account for vehicle stream characteristics as follows (northbound left turn illustrated):

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.02(2-1)} = 0.980$$

$$v_{NBL,pce} = \frac{v_{NBL}}{f_{HV}} = \frac{112}{0.980} = 114 \text{ pc/h}$$

The resulting adjusted flow rates for all movements accounting for Steps 1 and 2 are therefore computed as follows:



Step 3: Determine Circulating and Exiting Flow Rates

The circulating and exiting flows are calculated for each leg. For the south leg (northbound entry), the circulating flow is calculated as follows:

$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

$$v_{c,NB,pce} = 21 + 190 + 21 + 304 + 206 + 54 = 796 \text{ pc/h}$$

Similarly, $v_{c,SB,pce}$ = 769 pc/h, $v_{c,EB,pce}$ = 487 pc/h, and $v_{c,WB,pce}$ = 655 pc/h.

For this problem, one exit flow rate is needed: the northbound exit flow rate, which serves as the conflicting flow for the westbound bypass lane. Because all westbound right turns are assumed to use the bypass lane, they are excluded from the conflicting exit flow as follows:

$$v_{ex,pce,NB} = v_{SBU,pce} + v_{EBL,pce} + v_{NBT,pce} + v_{WBR,e,pce}$$

$$v_{ex,pce,NB} = 21 + 206 + 227 + 0 = 454 \text{ pc/h}$$

Step 4: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing the movement flow rates that enter the roundabout (without using a bypass lane). Because this is a single-lane roundabout, no lane-use calculations are needed.

The entry flow rates are calculated as follows, assuming that all right-turn volumes on the westbound and southbound approaches use the bypass lane provided and not the entry:

$$v_{e,NB,pce} = v_{NBU,pce} + v_{NBL,pce} + v_{NBT,pce} + v_{NBR,e,pce} = 33 + 114 + 227 + 54 = 428 \text{ pc/h}$$

$$v_{e,SB,pce} = v_{SBU,pce} + v_{SBL,pce} + v_{SBT,pce} + v_{SBR,e,pce} = 21 + 190 + 103 + 0 = 314 \text{ pc/h}$$

$$v_{e,EB,pce} = v_{EBU,pce} + v_{EBL,pce} + v_{EBT,pce} + v_{EBR,e,pce} = 54 + 206 + 304 + 92 = 656 \text{ pc/h}$$

$$v_{e,WB,pce} = v_{WBU,pce} + v_{WBL,pce} + v_{WBT,pce} + v_{WBR,e,pce} = 21 + 119 + 428 + 0 = 568 \text{ pc/h}$$

Step 5: Determine the Capacity of Each Entry Lane and Bypass Lane as Appropriate in Passenger Car Equivalents

By using the single-lane capacity equation (Equation 21-1), the capacity for each entry lane is given as follows:

$$c_{pce,NB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce,NB}} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(796)} = 510 \text{ pc/h}$$

$$c_{pce,SB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce,SB}} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(769)} = 524 \text{ pc/h}$$

$$c_{pce,EB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce,EB}} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(487)} = 694 \text{ pc/h}$$

$$c_{pce,WB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{c,pce,WB}} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(655)} = 587 \text{ pc/h}$$

By using the equation for a bypass lane opposed by a single exit lane (Equation 21-6), the capacity for the westbound bypass lane is given as follows:

$$c_{bypass,pce,WB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)v_{ex,pce,NB}} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(454)} = 718 \text{ pc/h}$$

Step 6: Determine Pedestrian Impedance to Vehicles

The south leg (northbound entry) has a conflicting pedestrian flow rate, n_{ped}, of 50 p/h. Therefore, the pedestrian impedance factor is calculated by using Exhibit 21-17 as follows:

$$f_{ped} = 1 - 0.000137n_{ped} = 1 - 0.000137(50) = 0.993$$

The other legs have negligible pedestrian activity and therefore have $f_{ped} = 1$.

Step 7: Convert Lane Flow Rates and Capacities into Vehicles per Hour

The capacity for a given lane is converted back to vehicles by first determining the heavy-vehicle adjustment factor for the lane and then multiplying it by the capacity in passenger car equivalents. For this example, since all turning movements on each entry have the same f_{HV}, each entry will also have the same f_{HV}, 0.980.

$$c_{NB} = c_{pce,NB}f_{HV,e,NB}f_{ped} = (510)(0.980)(0.993) = 497 \text{ veh/h}$$

$$c_{SB} = c_{pce,SB} f_{HV,e,SB} f_{ped} = (524)(0.980)(1) = 514 \text{ veh/h}$$

$$c_{EB} = c_{pce,EB} f_{HV,e,EB} f_{ped} = (694)(0.980)(1) = 680 \text{ veh/h}$$

$$c_{WB} = c_{pce,WB} f_{HV,e,WB} f_{ped} = (587)(0.980)(1) = 575 \text{ veh/h}$$

$$c_{\text{bypass},WB} = c_{\text{bypass},pce,NB} f_{HV,e,WB} f_{ped} = (718)(0.980)(1) = 704 \text{ veh/h}$$

Calculations for the entry flow rates are as follows:

$$v_{NB} = v_{pce,NB} f_{HV,e,NB} = (428)(0.980) = 420 \text{ veh/h}$$

$$v_{SB} = v_{pce,SB} f_{HV,e,SB} = (314)(0.980) = 308 \text{ veh/h}$$

$$v_{EB} = v_{pce,EB} f_{HV,e,EB} = (656)(0.980) = 643 \text{ veh/h}$$

$$v_{WB} = v_{pce,WB} f_{HV,e,WB} = (568)(0.980) = 557 \text{ veh/h}$$

$$v_{\text{bypass},WB} = v_{\text{bypass},pce,NB} f_{HV,e,WB} f_{ped} = (662)(0.980) = 649 \text{ veh/h}$$

Step 8: Compute the Volume-to-Capacity Ratio for Each Lane

The volume-to-capacity ratios for each entry lane are calculated as follows:

$$x_{NB} = \frac{420}{497} = 0.85$$

$$x_{SB} = \frac{308}{514} = 0.60$$

$$x_{EB} = \frac{643}{680} = 0.95$$

$$x_{WB} = \frac{557}{575} = 0.97$$

$$x_{bypass,WB} = \frac{649}{704} = 0.92$$

Step 9: Compute the Average Control Delay for Each Lane

The control delay for the northbound entry lane is computed as follows:

$$d_{NB} = \frac{3{,}600}{497} + 900(0.25)\left[0.85 - 1 + \sqrt{(0.85-1)^2 + \frac{\left(\frac{3{,}600}{497}\right)0.85}{450(0.25)}}\right]$$

$$+ 5 \times \min[0.85, 1] = 39.6 \text{ s/veh (assuming no rounding of } x\text{)}$$

Similarly, d_{SB} = 19.9 s, $d_{\text{bypass},SB}$ = 0 s (assumed), d_{EB} = 46.7 s, d_{WB} = 56.5 s, and $d_{\text{bypass},WB}$ = 41.5 s.

Step 10: Determine LOS for Each Lane on Each Approach

Using Exhibit 21-1, the LOS for each lane is determined as follows:

Lane	Control Delay (s/veh)	LOS
Northbound entry	39.6	E
Southbound entry	19.9	C
Southbound bypass lane	0 (assumed)	A
Eastbound entry	46.7	E
Westbound entry	56.5	F
Westbound bypass lane	41.5	E

Step 11: Compute the Average Control Delay and Determine LOS for Each Approach and the Roundabout as a Whole

The control delays for the northbound and eastbound approaches are equal to the control delay for the entry lanes, as both of these approaches have only one lane. On the basis of Exhibit 21-1, these approaches are both assigned LOS E.

The control delay calculations for the westbound and southbound approaches include the effects of their bypass lanes as follows:

$$d_{WB} = \frac{(56.5)(557) + (41.5)(649)}{557 + 649} = 48.4 \text{ s/veh}$$

$$d_{SB} = \frac{(19.9)(308) + (0.0)(617)}{308 + 617} = 6.6 \text{ s/veh}$$

On the basis of Exhibit 21-1, these approaches are respectively assigned LOS E and LOS A.

Similarly, intersection control delay is computed as follows:

$$d_{\text{intersection}} = \frac{(39.6)(420) + (6.6)(925) + (46.7)(643) + (48.4)(1206)}{420 + 925 + 643 + 1206} = 34.8 \text{ s/veh}$$

On the basis of Exhibit 21-1, the intersection is assigned LOS D.

Step 12: Compute 95th Percentile Queues for Each Lane

The 95th percentile queue is computed for each lane. An example calculation for the northbound entry is given as follows:

$$Q_{95,NB} = 900(0.25)\left[0.85 - 1 + \sqrt{(1-0.85)^2 + \frac{\left(\frac{3{,}600}{497}\right)0.85}{150(0.25)}}\right]\left(\frac{497}{3{,}600}\right) = 8.6 \text{ veh}$$

For design purposes, this value is typically rounded up to the nearest vehicle, which for this case would be 9 veh.

Similarly, $Q_{95,SB} = 3.9$ veh, $Q_{95,EB} = 13.4$ veh, $Q_{95,WB} = 13.3$ veh, and $Q_{95,\text{bypass},WB} =$ 12.5 veh.

Discussion

The results indicate that the overall roundabout is operating at LOS D based on a control delay very close to the boundary between LOS D and LOS E. However, three approaches (northbound, eastbound, and westbound) are operating at LOS E, and one lane (westbound entry) is operating at LOS F (based on control delay). In addition, two of the four entries have volume-to-capacity ratios exceeding 0.95 during the peak 15 min of the hour analyzed. If the performance standard for this intersection were LOS D, three approaches would not meet the standard, even though the overall intersection meets the standard. For these reasons, the analyst should report volume-to-capacity ratios, control delay, and queue lengths for each lane, in addition to the aggregated measures, for a more complete picture of operational performance.

The analyst should be careful not to mask key operational performance issues by reporting overall intersection performance without also reporting the performance of each lane, or at least the worst-performing lane.

EXAMPLE PROBLEM 2: MULTILANE ROUNDABOUT

The Facts

This is also an example of an operational analysis, despite the fact that lane utilization data are unknown and must be assumed.

The following data are available to describe the traffic and geometric characteristics of this location:

- Percent heavy vehicles for eastbound and westbound movements = 5%,
- Percent heavy vehicles for northbound and southbound movements = 2%,
- Peak hour factor = 0.95,
- Negligible pedestrian activity, and
- Volumes and lane configurations as shown in Exhibit 21-24.

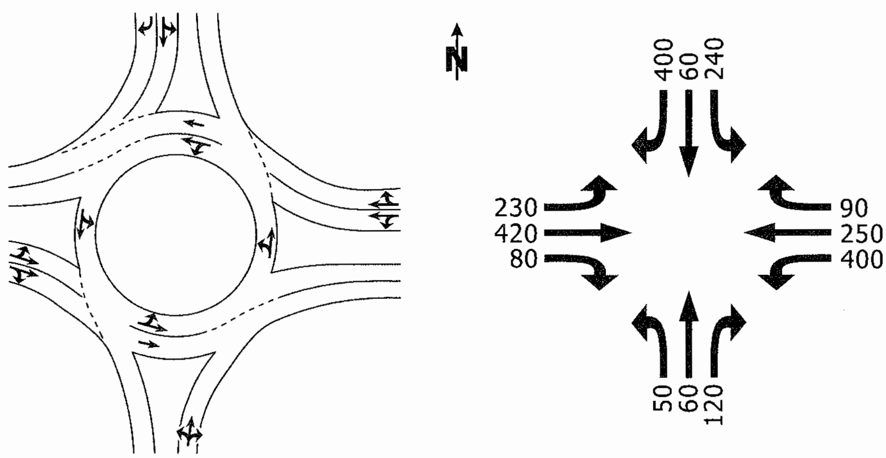


Exhibit 21-24
Demand Volumes and Lane Configurations for Example Problem 2

Comments

Lane use is not specified for the eastbound and westbound approaches; therefore, the percentage flow in the right lane is assumed to be 53%, per Exhibit 21-21.

Step 1: Convert Movement Demand Volumes to Flow Rates

Each turning-movement demand volume given in the problem is converted to a demand flow rate by dividing by the peak hour factor. As an example, the eastbound left demand volume is converted to a demand flow rate as follows:

$$v_{EBL} = \frac{V_{EBL}}{PHF} = \frac{230}{0.95} = 242 \text{ veh/h}$$

Step 2: Adjust Flow Rates for Heavy Vehicles

The heavy-vehicle adjustment factor for the eastbound and westbound movements is calculated as follows:

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} = \frac{1}{1+0.05(2-1)} = 0.952$$

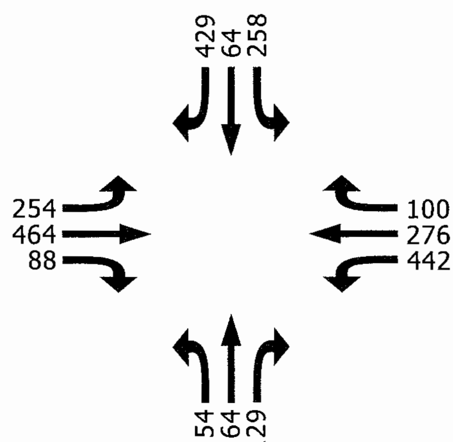
Similarly, the heavy-vehicle adjustment factor for the northbound and southbound movements is calculated as follows:

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} = \frac{1}{1+0.02(2-1)} = 0.980$$

This is applied to each movement as follows (eastbound left turn illustrated):

$$v_{EBL,pce} = \frac{v_{EBL}}{f_{HV}} = \frac{242}{0.952} = 254 \text{ pc/h}$$

The resulting adjusted flow rates for all movements, accounting for Steps 1 and 2, are therefore as follows:



Step 3: Determine Circulating and Exiting Flow Rates

For this problem, only circulating flows need to be calculated for each leg. For the west leg (eastbound entry), the circulating flow is calculated as follows:

$$v_{c,EB,pce} = v_{NBU,pce} + v_{WBL,pce} + v_{WBU,pce} + v_{SBT,pce} + v_{SBL,pce} + v_{SBU,pce}$$

$$v_{c,EB,pce} = 0 + 442 + 0 + 64 + 258 + 0 = 764 \text{ pc/h}$$

Similarly, $v_{c,WB,pce} = 372$ pc/h, $v_{c,NB,pce} = 976$ pc/h, and $v_{c,SB,pce} = 772$ pc/h.

Step 4: Determine Entry Flow Rates by Lane

The entry flow rate is calculated by summing up the movement flow rates that enter the roundabout. This problem presents four unique cases.

- *Northbound*: The northbound entry has only one lane. Therefore, the entry flow is simply the sum of the movements, or 54 + 64 + 129 = 247 pc/h.
- *Southbound*: The southbound entry has two lanes: a shared through–left lane and a right-turn-only lane. Therefore, the flow rate in the right lane is simply the right-turn movement flow, or 429 pc/h, and the flow rate in the left lane is the sum of the left-turn and through movements, or 258 + 64 = 322 pc/h.
- *Eastbound*: The eastbound entry has shared left–through and through–right lanes. A check is needed to determine whether any de facto lanes are in effect. These checks are as follows:
  - *Left lane*: The left-turn flow rate, 254 pc/h, is less than the sum of the through and right-turn flow rates, 464 + 88 = 552 pc/h. Therefore, some of the through volume is assumed to use the left lane, and no de facto left-turn lane condition is present.
  - *Right lane*: The right-turn flow rate, 88 pc/h, is less than the sum of the left-turn and through flow rates, 254 + 464 = 718 pc/h. Therefore, some of the through volume is assumed to use the right lane, and no de facto right-turn lane condition is present.
  - The total entry flow (254 + 464 + 88 = 806 pc/h) is therefore distributed over the two lanes, with flow biased to the right lane using the assumed lane-use factor identified previously:
    - Right lane: (806)(0.53) = 427 pc/h
    - Left lane: 806 – 427 = 379 pc/h
- *Westbound*: The westbound entry also has shared left–through and through–right lanes, and so a similar check is needed for de facto lanes. The left-turn flow rate, 442 pc/h, is greater than the sum of the through and right-turn flow rates, 276 + 100 = 376 pc/h. Therefore, the left lane is assumed to operate as a de facto left-turn lane. Therefore, the left-lane flow rate is equal to the left-turn flow rate, or 442 pc/h, and the right-lane flow rate is equal to the sum of the through- and right-turn-movement flow rates, or 376 pc/h.

Step 5: Determine the Capacity of Each Entry Lane and Bypass Lane as Appropriate in Passenger Car Equivalents

The capacity calculations for each approach are calculated as follows:

- *Northbound*: The northbound entry is a single-lane entry opposed by two circulating lanes. Therefore, Equation 21-3 is used as follows:

$$c_{pce,NB} = 1{,}130e^{(-0.7\times10^{-3})(976)} = 571 \text{ pc/h}$$

- *Southbound*: The southbound entry is a two-lane entry opposed by two circulating lanes. Therefore, Equation 21-4 is used for the right lane, and Equation 21-5 is used for the left lane:

$$c_{pce,SB,R} = 1{,}130e^{\left(-0.7\times10^{-3}\right)(772)} = 658 \text{ pc/h}$$

$$c_{pce,SB,L} = 1{,}130e^{\left(-0.75\times10^{-3}\right)(772)} = 633 \text{ pc/h}$$

- *Eastbound*: The eastbound entry is a two-lane entry opposed by one circulating lane. Therefore, the capacity for each lane is calculated by using Equation 21-2 as follows:

$$c_{pce,EB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(764)} = 526 \text{ pc/h}$$

- *Westbound*: The westbound entry is also a two-lane entry opposed by one circulating lane, so its capacity calculation is similar to that for the eastbound entry:

$$c_{pce,WB} = 1{,}130e^{\left(-1.0\times10^{-3}\right)(372)} = 779 \text{ pc/h}$$

Step 6: Determine Pedestrian Impedance to Vehicles

For this problem pedestrians have been assumed to be negligible, so no impedance calculations are performed.

Step 7: Convert Lane Flow Rates and Capacities into Vehicles per Hour

The capacity for a given lane is converted back to vehicles by first determining the heavy-vehicle adjustment factor for the lane and then multiplying it by the capacity in passenger car equivalents. For this example, since all turning movements on the eastbound and westbound entries have the same f_{HV}, each of the lanes on the eastbound and westbound entries can be assumed to have the same f_{HV}, 0.952.

$$c_{EB,R} = c_{pce,EB,R} f_{HV,e,EB} = (526)(0.952) = 501 \text{ veh/h}$$

Similarly, $c_{EB,L}$ = 501 veh/h, $c_{WB,L}$ = 742 veh/h, and $c_{WB,R}$ = 742 veh/h.

Since all turning movements on the northbound and southbound entries have the same f_{HV}, each of the lanes on those entries can be assumed to have the same f_{HV}, 0.980.

$$c_{NB} = c_{pce,NB} f_{HV,e,NB} = (571)(0.980) = 560 \text{ veh/h}$$

Similarly, $c_{SB,L}$ = 621 veh/h and $c_{SB,R}$ = 645 veh/h.

Calculations for the entry flow rates are as follows:

$$v_{EB,R} = v_{pce,EB,R} f_{HV,e,EB} = (427)(0.952) = 407 \text{ veh/h}$$

$$v_{NB} = v_{pce,NB} f_{HV,e,NB} = (247)(0.980) = 242 \text{ veh/h}$$

Similarly, $v_{EB,L}$ = 361 veh/h, $v_{WB,L}$ = 421 veh/h, $v_{WB,R}$ = 358 veh/h, $v_{SB,L}$ = 316 veh/h, and $v_{SB,R}$ = 421 veh/h.

Step 8: Compute the Volume-to-Capacity Ratio for Each Lane

The volume-to-capacity ratio for each lane is calculated as follows:

$$x_{NB} = 242/560 = 0.43$$

$$x_{SB,L} = 316/621 = 0.51$$

$$x_{SB,R} = 421/645 = 0.65$$

$$x_{EB,L} = 361/501 = 0.72$$

$$x_{EB,R} = 407/501 = 0.81$$

$$x_{WB,L} = 421/742 = 0.57$$

$$x_{WB,R} = 358/742 = 0.48$$

Step 9: Compute the Average Control Delay for Each Lane

The control delay for the northbound entry lane is computed as follows:

$$d_{NB} = \frac{3{,}600}{560} + 900(0.25)\left[\frac{242}{560} - 1 + \sqrt{\left(\frac{242}{560} - 1\right)^2 + \frac{\left(\frac{3{,}600}{560}\right)\frac{242}{560}}{450(0.25)}}\right]$$

$$+ 5 \times \min\left[\frac{242}{560}, 1\right] = 13.4 \text{ s/veh}$$

Similarly, $d_{SB,L} = 14.2$ s, $d_{SB,R} = 18.7$ s, $d_{EB,L} = 27.2$ s, $d_{EB,R} = 35.4$ s, $d_{WB,L} = 13.9$ s, and $d_{WB,R} = 11.7$ s.

Step 10: Determine LOS for Each Lane on Each Approach

On the basis of Exhibit 21-1, the LOS for each lane is determined as follows:

Critical Lane	Control Delay (s/veh)	LOS
Northbound entry	13.4	B
Southbound left lane	14.2	B
Southbound right lane	18.7	C
Eastbound left lane	27.2	D
Eastbound right lane	35.4	E
Westbound left lane	13.9	B
Westbound right lane	11.7	B

Step 11: Compute the Average Control Delay and Determine LOS for Each Approach and the Roundabout as a Whole

The control delay for the northbound approaches is equal to the control delay for the entry lane, 13.4 s, as the approach has only one lane. The control delays for the other approaches are as follows:

$$d_{SB} = \frac{(14.2)(316) + (18.7)(421)}{316 + 421} = 16.8 \text{ s/veh}$$

$$d_{EB} = \frac{(27.2)(361) + (35.4)(407)}{361 + 407} = 31.5 \text{ s/veh}$$

$$d_{WB} = \frac{(13.9)(421) + (11.7)(358)}{421 + 358} = 12.9 \text{ s/veh}$$

On the basis of Exhibit 21-1, these approaches are respectively assigned LOS B, LOS C, LOS D, and LOS B.

Similarly, control delay for the intersection is computed as follows:

$$d_{\text{intersection}} = \frac{(13.4)(242) + (16.8)(736) + (31.5)(768) + (12.9)(779)}{242 + 736 + 768 + 779} = 19.7 \text{ s/veh}$$

On the basis of Exhibit 21-1, the intersection is assigned LOS C.

Step 12: Compute 95th Percentile Queues for Each Lane

The 95th percentile queue is computed for each lane. An example calculation for the northbound entry is given as follows:

$$Q_{95,NB} = 900(0.25)\left[\frac{242}{560} - 1 + \sqrt{\left(1 - \frac{242}{560}\right)^2 + \frac{\left(\frac{3{,}600}{560}\right)\left(\frac{242}{560}\right)}{150(0.25)}}\right]\left(\frac{560}{3{,}600}\right) = 2.2 \text{ veh}$$

For design purposes, this value is typically rounded up to the nearest vehicle, in this case 3 veh.

Discussion

The results indicate that the intersection as a whole operates at LOS C on the basis of control delay during the peak 15 min of the analysis hour. However, the eastbound approach operates at LOS D, and the right lane of that approach operates at LOS E (with a control delay very close to the boundary of LOS D and LOS E) and with a volume-to-capacity ratio of 0.81. The analyst should report both the overall performance and those of the individual lanes to provide a more complete picture of operational performance.

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CHAPTER 22
INTERCHANGE RAMP TERMINALS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Interchange ramp terminals are critical components of the highway network. They provide the connection between various highway facilities (i.e., freeway–arterial, arterial–arterial, etc.), and thus their efficient operation is essential. Interchanges have to be designed to work in harmony with the freeway, the ramps, and the arterials. In addition, they need to provide adequate capacity to avoid affecting the connecting facilities.

SCOPE OF THE CHAPTER

Chapter 22, Interchange Ramp Terminals, presents the methodology for the analysis of interchanges involving freeways and surface streets (i.e., service interchanges), and it was developed primarily on the basis of research conducted through the National Cooperative Highway Research Program (*1–3*) and elsewhere (*4*).

The methodology addresses interchanges with signalized intersections, interchanges with roundabouts, and the impact and operations of adjacent closely spaced intersections. Interchanges with two-way STOP-controlled intersections or interchanges consisting of a signalized intersection and a roundabout cannot be evaluated with the procedures of this chapter. Traffic circles (e.g., intersections with a circular island in the middle and signals at the approaches) are not considered in this chapter. The scope of this chapter includes the operational analysis for a full range of service interchange types, including diamond, partial cloverleaf (parclo), and single-point urban interchanges (SPUIs). It also includes a methodology for assessing the operational performance of various types of interchanges for purposes of interchange type selection. The chapter can be used to obtain guidance for assessing various interchange types with respect to their operational performance; it does not provide guidance for selecting an appropriate interchange type with respect to economic, environmental, land use, and other such concerns. The methodology addresses at-grade intersections, not including the freeway proper, and focuses on surface streets; it does not analyze freeway-to-freeway interchanges.

LIMITATIONS OF THE METHODOLOGY

The methodology does not address oversaturated conditions, particularly cases when the downstream queue spills back into the upstream intersection (i.e., when the internal queue exceeds the available storage of the link). It does not address spillback from inadequate turning pocket length. It does not explicitly evaluate the impact of spillback on freeway operations; however, it does estimate the expected queue on the ramps. It does not consider the impacts of ramp metering and spillback from the freeway into the interchange. The method does not estimate lane utilizations for cases when one or both intersections contain an approach that is not part of the prescribed interchange configuration given in the Types of Interchanges section; however, guidance is provided for addressing those cases. The methodology does not specifically address diverging interchanges or continuous flow interchanges (there is limited information with regard to the operation of these emerging interchange types, but it is likely that future research will focus on developing methods for evaluating their operational performance). The methodology provides delay estimates but does not provide link travel times and speeds.

In cases where the user is interested in the analysis of conditions that fall under the above methodological limitations or in the investigation of dynamic traffic operations (i.e., those that evolve in time and space), the use of another analysis tool, such as simulation modeling, is advised. Section 3 includes information on the use of alternative tools for the analysis of interchange ramp terminals.

The operational analysis is only one factor to be considered in the design or redesign of an interchange ramp terminal. Other important factors include right-of-way availability and economic and environmental constraints. The scope of this chapter does not include such considerations; the chapter focuses only on the traffic operational performance of signalized interchanges.

TYPES OF INTERCHANGES

A number of different types of interchanges are recognized in the literature. *A Policy on Geometric Design of Highways and Streets* (5) provides extensive information on interchange designs and their characteristics. This section illustrates and discusses the interchange designs considered in this chapter, namely diamond interchanges, parclos, SPUIs, and interchanges with roundabouts.

Diamond Interchanges

Most forms of diamond interchanges result in two or more closely spaced surface intersections, as illustrated in Exhibit 22-1. On a diamond interchange, only one connection is made for each freeway entry and exit, with one connection per quadrant. Left- and right-turning movements are used for entry to or exit from the two directions of the surface facility, which necessitates left-turning movements. When demands are low (generally in rural areas), the junction of diamond interchange ramps with the surface facility is typically controlled by stop or yield signs. If traffic demands are sufficiently high, signalization becomes necessary.

There are many variations of the diamond interchange. The typical diamond configuration has three subcategories defined by the spacing of the intersections formed by the ramp–street connections. Conventional diamond interchanges provide a separation of 800 ft or more between the two intersections. Compressed diamond interchanges have intersections spaced between 400 and 800 ft, and tight urban diamond interchanges feature spacing of less than 400 ft. Because of right-of-way constraints, compressed diamonds are more likely to be used in urban areas, while conventional diamond interchanges are more likely to be used in rural or suburban settings.

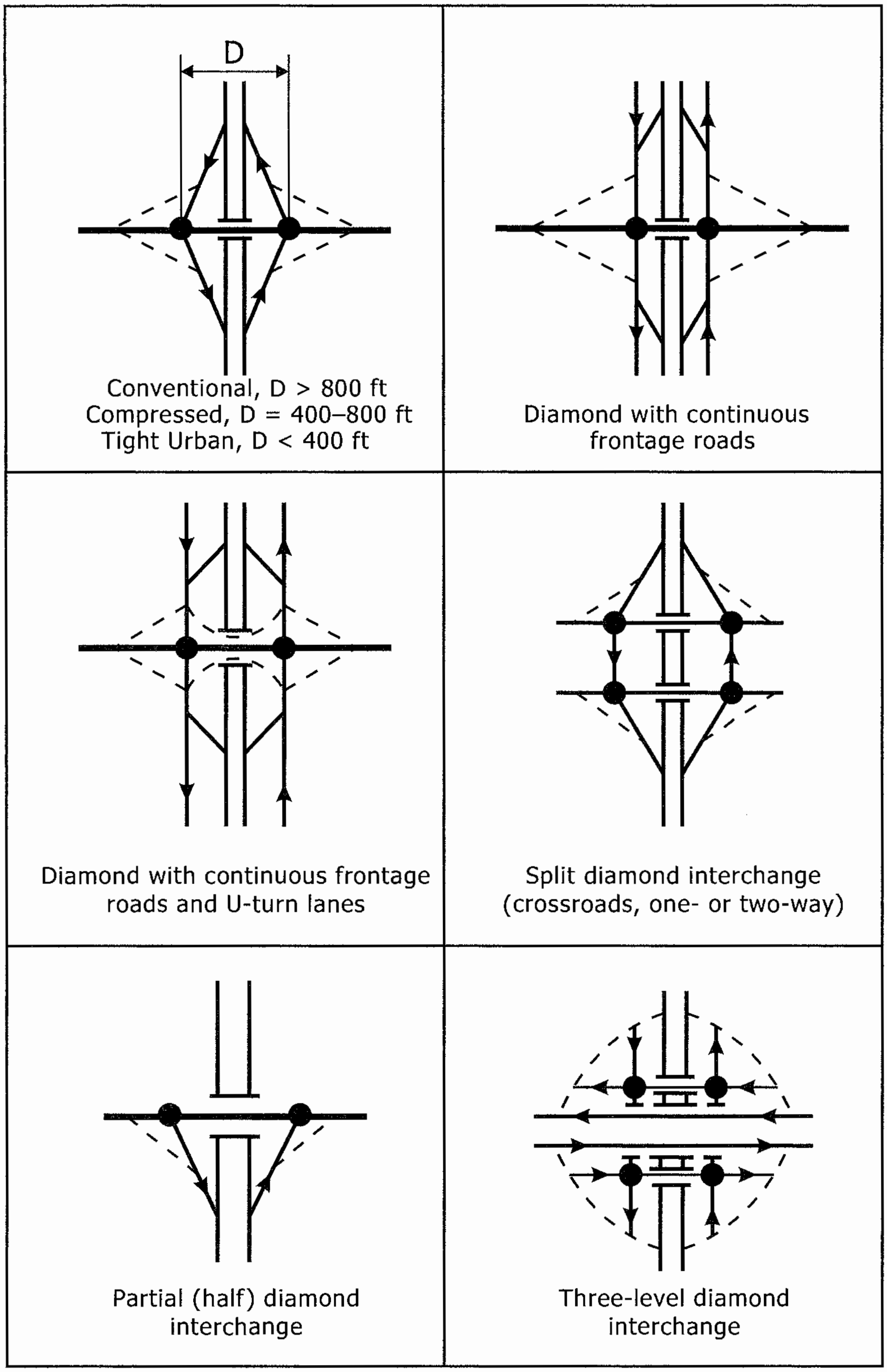


Exhibit 22-1
Types of Diamond Interchanges

Note: — — — — Possible alternative configuration of signal bypasses operating as unsignalized movements; these are movements that are not using the ramp terminals.

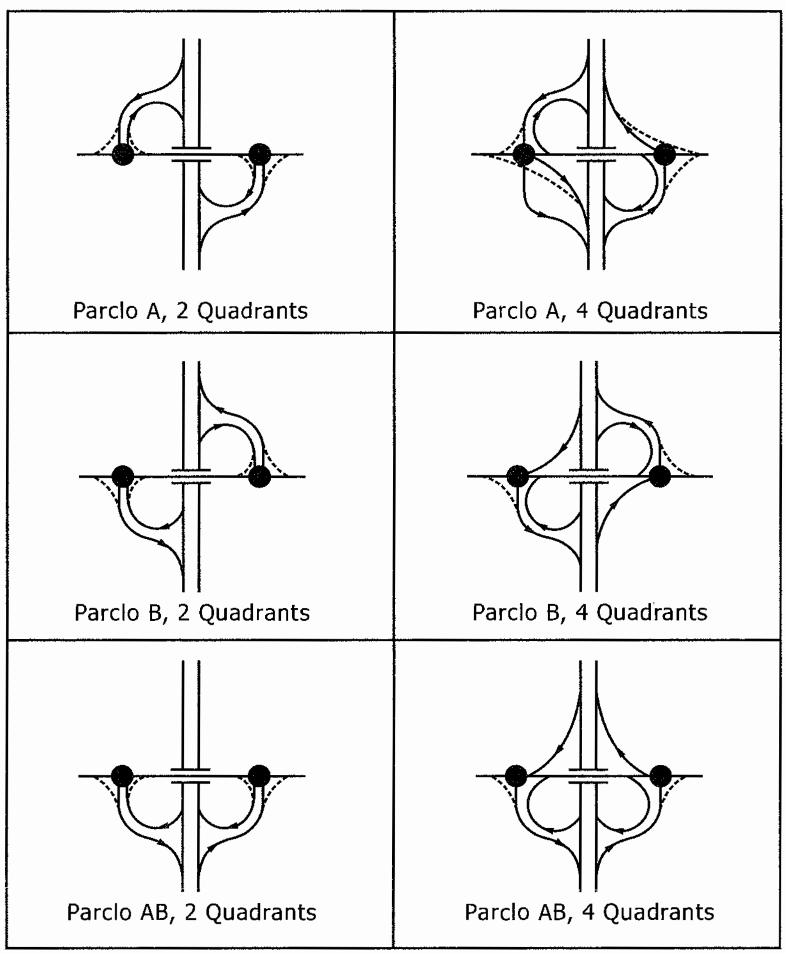
Split diamond interchanges have freeway entry and exit ramps separated at the street level, creating four intersections. Diamond configurations also can be combined with continuous one-way frontage roads. The frontage roads become one-way arterials, and turning movements at the intersections created by the diamond interchange become even more complex. Separated U-turn roadways may be added, removing U-turns from the signal scheme, if there is a signal. A partial diamond interchange has fewer than four ramps, and not all freeway–street or street–freeway movements are served. A three-level diamond interchange features two divided levels, so that ramps are necessary on both facilities to allow continuous through movements.

All these forms of diamond interchanges are depicted in Exhibit 22-1. The methodology in this chapter is applicable to all diamond interchange forms except the split diamond and the three-level diamond. The methodology addresses interchanges where both terminals are signalized or both terminals are roundabouts.

Parclo Interchanges

Parclo interchanges are depicted in Exhibit 22-2. A variety of parclo interchanges can be created with one or two loop ramps. In such cases, one or two of the outer ramps take the form of a diamond ramp, allowing a movement to take place by means of a right turn. In some parclo configurations, left turns also may be made onto or off of a loop ramp. The methodology in this chapter is applicable to parclo interchanges where both terminals are signalized or both terminals are roundabouts.

Exhibit 22-2
Types of Parclo Interchanges

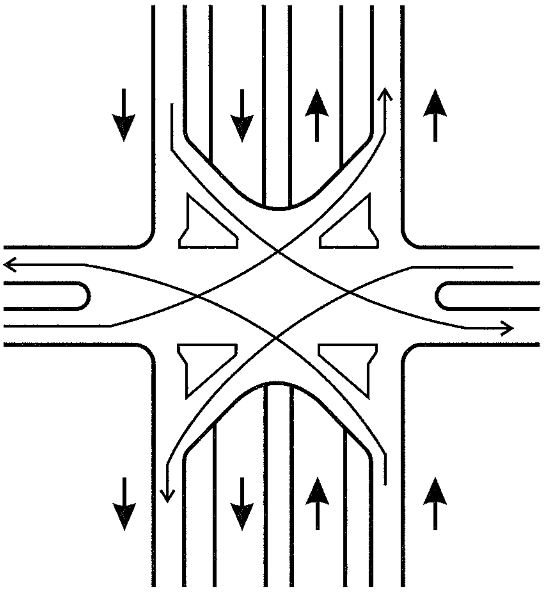


Note: – – – – Possible alternative configuration of signal bypasses operating as unsignalized movements; these are movements that are not using the ramp terminals.

Single-Point Urban Interchanges

A SPUI combines all the ramp movements into a single signalized intersection and has the advantage of operating as such. The design eliminates the critical issue of coordinating the operation of two closely spaced intersections. The SPUI is depicted in Exhibit 22-3.

Exhibit 22-3
Single-Point Urban Interchange

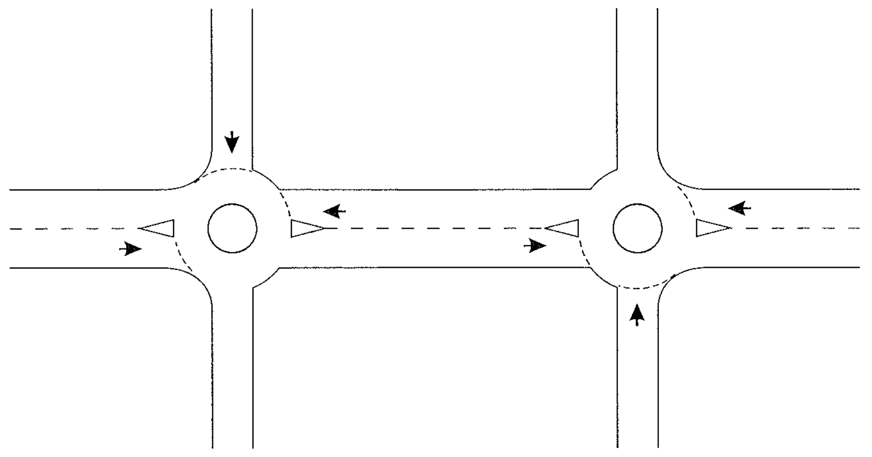


Interchanges with Roundabouts

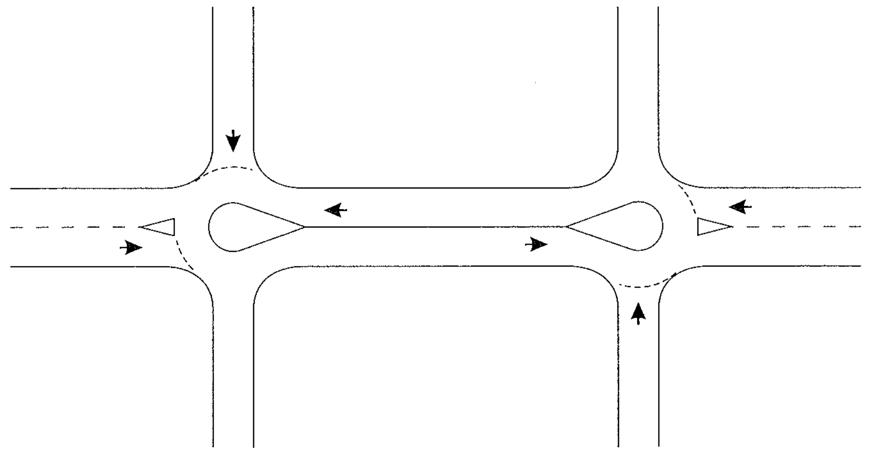
Roundabout intersections can replace signalized or stop-controlled intersections as interchange ramp terminals. Three types of roundabout intersection designs are typically used in the United States and are illustrated in Exhibit 22-4. The first design consists of two traditional roundabouts at the two nodes of the interchange. The second design is called the raindrop roundabout interchange, and it restricts certain movements within each roundabout by creating raindrop-shaped central islands. The two designs are essentially the same, except that the first should be provided when U-turns are allowed or when there is an additional approach to the roundabout. The last design consists of a single roundabout spanning both sides of the freeway via over- or underpasses.

These three designs are applicable to both diamond and parclo interchanges. Their major advantage is that they can reduce the number of lanes needed between terminals, thereby significantly reducing structure-related costs. They also eliminate the need for coordinating signal operations at the two closely spaced intersections. A potential disadvantage of using roundabouts is that spillback from a downstream facility into the roundabout may result in gridlock for all movements at the roundabout, since all movements must use the circulating roadway.

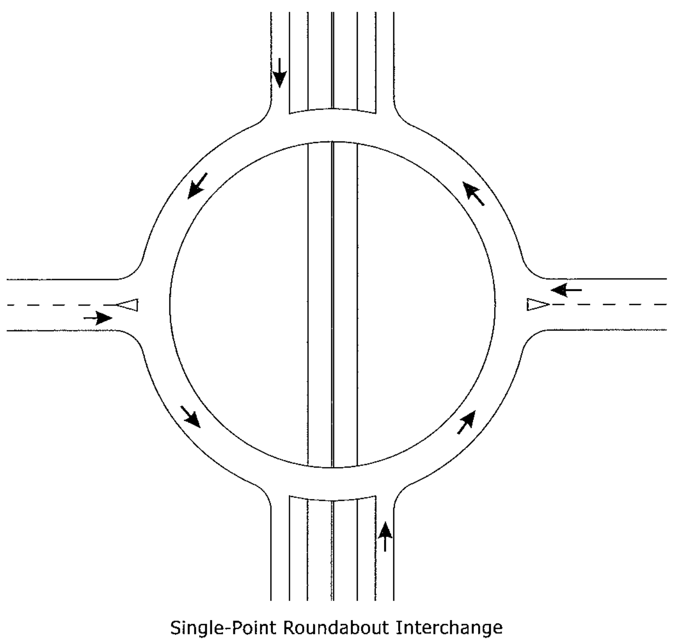
Exhibit 22-4
Interchanges with Roundabouts



Diamond Interchange with Circular Ramp Terminals



Diamond Interchange with Raindrop-Shaped Ramp Terminals



Single-Point Roundabout Interchange

UNIQUE OPERATIONAL CHARACTERISTICS OF INTERCHANGES

Influence of Interchange Type on Turning Movements

The type of interchange has a major influence on turning movements. Movements that involve a right-side merge in one configuration become left turns in another. Movements approaching the interchange on the surface facility are also affected by the interchange type, depending on whether the ramp movements involve left or right turns. Thus, the lane utilization of the external approaches to the interchange varies as a function of the type of interchange and the relative proportion of the turning movements at the downstream intersection.

In selecting an appropriate type of interchange, the impacts on the turning movements should be considered. Left-turning movements are always the most difficult in terms of efficiency of operation, and high-volume left-turning movements should be avoided, if possible. By selecting a type of interchange that requires left turns only for low-demand movements, the overall operation can be enhanced significantly. However, it is not always possible to accomplish this. Right-of-way limitations or agency policies may preclude the use of loop ramps, and economic and environmental constraints may make multilevel structures undesirable.

Because of the influence of interchange type on turning movements, and to be able to compare various interchange types, their level of service (LOS) is based on origin–destination (O-D) demands through the interchange, which are identical regardless of the interchange type. The methodology in this chapter uses both O-D demands and turning movement demands; one set of demands can be derived from the other. Exhibit 22-5 illustrates the O-D demands at an interchange and gives their respective notation. To simplify the mapping process, it is assumed that the freeway is oriented north-south (N-S) and the surface arterial east-west (E-W). Note that Demands K and L indicate the through movement that travels across the surface street, not the freeway through traffic. These demands are expected to be minimal; however, they are included in the methodology because they constitute allowable movements through the interchange. Exhibit 22-6 illustrates the routing of all demands through a diamond interchange, while Exhibit 22-7 illustrates their routing through a Parclo AB-2Q. As shown, the O-D Demand E, eastbound from the arterial and northbound into the freeway, is a left turn in the diamond configuration, while it becomes a right turn in Parclo AB-2Q.

Exhibit 22-5
Illustration and Notation of O-D Demands at an Interchange

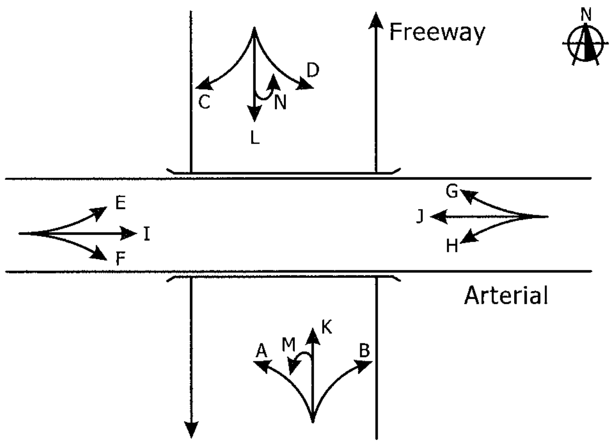


Exhibit 22-6
Illustration and Notation of O-D Demands at a Diamond Interchange

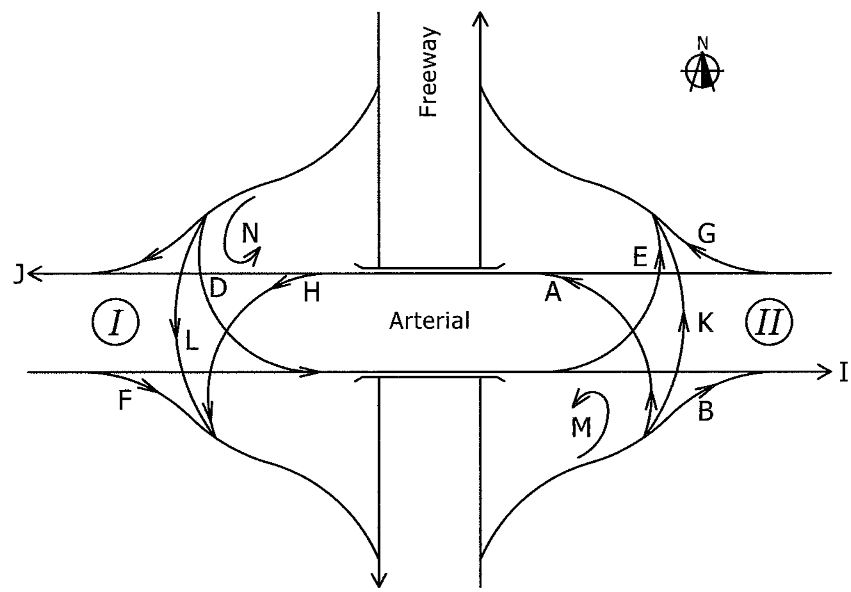
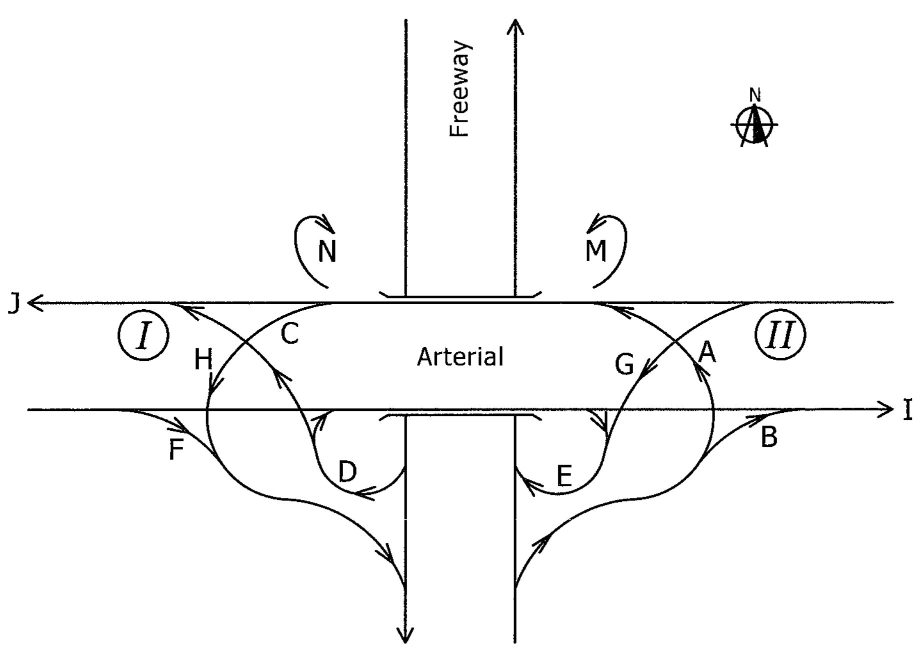


Exhibit 22-7
Illustration of O-D Demands Through a Parclo AB-2Q Interchange



Operational Effects of the Intersection Spacing

Those configurations with two closely spaced signalized intersections present unique challenges because the two intersections do not operate in isolation. Examples of such configurations include the diamond interchanges and two-quadrant parclos. The distance separating the two intersections limits the amount of storage available for queued vehicles. Also, the presence of a downstream queue may reduce or completely block the discharge from the upstream intersection.

Queuing at the downstream intersection can have one of the following two impacts on the discharge from the upstream intersection:

1. Queuing conditions at the downstream intersection are not severe enough to affect the upstream intersection; or
2. Queues at the downstream intersection reduce the rate of discharge upstream because of proximity of the back of the queue. When that occurs, portions of the upstream green cannot be used because of downstream blockage.

Queued vehicles within a short segment (or link) limit the effective length of the link, and vehicles can travel freely only from the upstream stop line to the back of the downstream queue. Because this distance may be small, the impact on the upstream discharge rate is significant. In this methodology, the effects of the presence of a queue at the downstream link are considered by estimating the amount of additional lost time experienced at the upstream intersection. The additional lost time is calculated as a function of the distance to the downstream queue at the beginning of the green for each of the upstream phases.

The extent of queuing at the downstream intersection depends on several factors, including the signal control at the upstream and downstream signals, the number and use of lanes at both intersections, and the upstream flow rates that feed the downstream intersection.

Some of these effects may also exist at locations where signalized intersections are closely spaced, particularly where heavy left-turn movements exist. This chapter addresses the interactions of interchange operations with those of adjacent closely spaced signalized intersections. Furthermore, the principles described in this chapter may be applied to similar situations in which closely spaced signalized intersections (other than those at interchanges) interact.

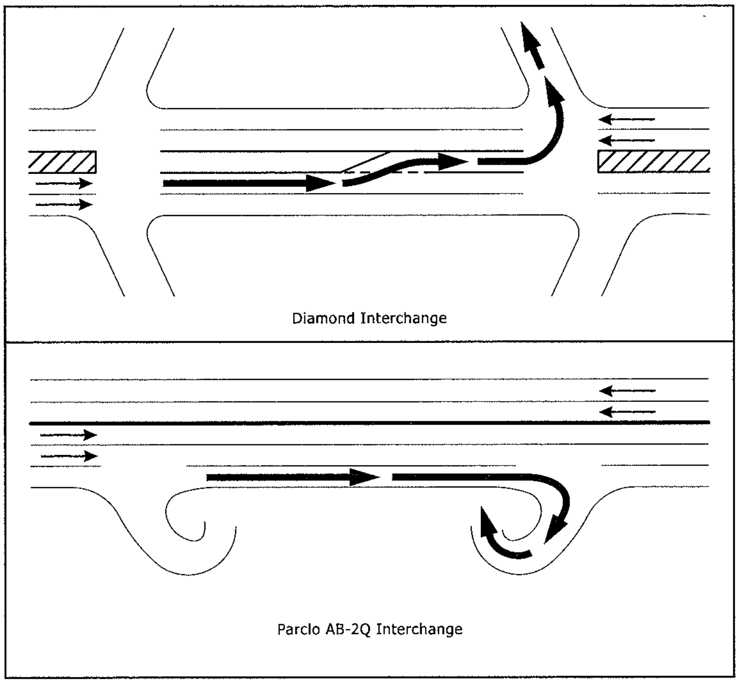
Similar issues may exist at interchanges with roundabouts that are near signalized intersections. When the queue from the signalized intersection reaches the roundabout, it might cause complete gridlock, since all movements through the roundabout must use the circulating roadway.

Lane Utilization for the External Through Movements

For two-intersection signalized interchanges, the lane utilization for the external through movements approaching the interchange on the surface facility is significantly affected by the direction and demand of the turning movements at the downstream intersection. As shown in Exhibit 22-8, high-volume downstream left turns will gravitate toward the left-side lanes at the upstream intersection, while the remaining through and right-turning vehicles will tend toward the right. Conversely, heavy-volume downstream right turns will gravitate toward the right side at the upstream intersection. This can create lane-use imbalances that exceed those at single intersections.

The methodology in this chapter identifies the highest-utilization lane at each of the upstream external through movements as a function of the interchange type, the number of through lanes, the distance between the two intersections, and the O-D demands.

Exhibit 22-8
Impact of Interchange Type on Lane Utilization



This chapter also considers the lane utilization of the arterial approaches at intersections adjacent to the interchange. Lane utilization at those intersections may be affected by turning movement demands at the interchange.

Demand Starvation

Demand starvation occurs when a signalized approach has adequate capacity but a significant portion of the traffic demand is held upstream because of the signalization pattern. For two-intersection signalized interchanges, 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. Thus, portions of the downstream green are unused while demand is stuck at the upstream intersection. Exhibit 22-9 illustrates the concept of demand starvation for an interchange. As shown, the internal left turn in the eastbound direction is green, blocking all westbound vehicles from reaching the westbound internal link green. Thus, demand starvation is experienced by the internal westbound through movement, where the signal is green, while the demand for it is blocked upstream.

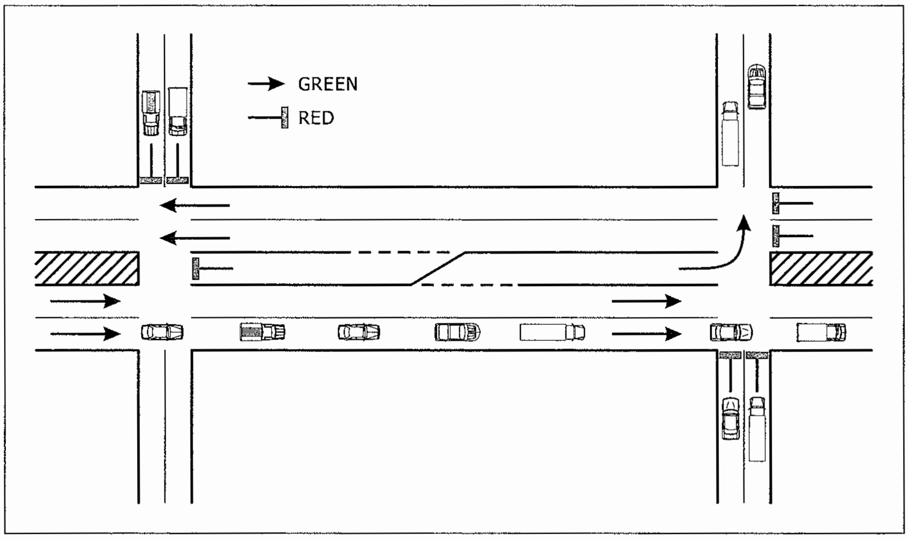


Exhibit 22-9
Demand Starvation at the Internal Link of a Diamond Interchange

LOS FRAMEWORK

Signalized Interchanges

The LOS designation is based on the operational performance of O-D demands (shown in Exhibit 22-5) through the interchange. The LOS for each O-D is based on the total average control delay experienced by that demand as it travels through the interchange. For example, for the diamond interchange shown in Exhibit 22-10, the delay for the O-D Movement H is equal to the sum of the average control delays (d_{WBTH}, d_{WBL}) at each of the lane group flows v_{WBTH}, v_{WBL} along its path. Thus, the delay d_W for O-D H is as follows:

Equation 22-1

$$d_w = d_{WBTH} + d_{WBL}$$

where d_{WBTH} is average control delay of the external westbound through movement (s/veh) and d_{WBL} is average control delay of the internal westbound left movement (s/veh).

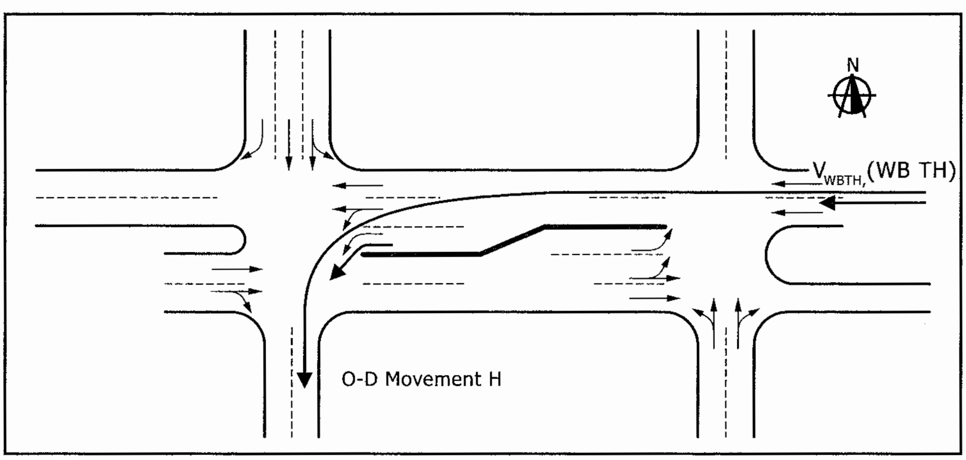


Exhibit 22-10
Illustration of the LOS Concept at a Diamond Interchange

Furthermore, LOS F is defined to occur when either the volume-to-capacity ratio (v/c) or the average queue-to-storage ratio (R_Q) for any of the lane groups that contain this O-D exceed 1, where R_Q refers to the average per lane queue storage ratio within the lane group. Storage is defined as the distance available for queued vehicles on a particular movement, and it is provided on a per lane basis. For example, if the left-turning lane group shown in Exhibit 22-10 has $v/c >$ 1, then the LOS for the entire O-D Movement H will be LOS F. If a particular lane group has $v/c > 1$, then all O-Ds that travel through this lane group will operate in LOS F, regardless of their delay. Similarly, if the average per lane queue in a particular lane group exceeds its available storage, then all O-Ds traveling through this lane group will operate in LOS F, regardless of their delay.

Exhibit 22-11 summarizes the LOS criteria for each O-D of an interchange. The values presented in Exhibit 22-11 are greater than those for signalized intersections by a factor of 1.5 to reflect the fact that some of the O-D movements would travel through two intersections, while others would travel through only one.

Exhibit 22-11
LOS Criteria for Each O-D of a Signalized Interchange

Control Delay (s/veh)	O-D LOS $v/c < 1$ and $R_Q < 1$ for Every Lane Group	 $v/c > 1$ for Any Lane Group	 $R_Q > 1$ for Any Lane Group
≤15	A	F	F
>15–30	B	F	F
>30–55	C	F	F
>55–85	D	F	F
>85–120	E	F	F
>120	F	F	F

Interchanges with Roundabouts

Similar to signalized interchanges, the LOS designation for interchanges with roundabouts is based on the operational performance of O-D demands through the interchange. The LOS for each O-D is based on the total average control delay experienced by that demand as it travels through the interchange. For example, for the interchange shown in Exhibit 22-12, the delay for the O-D Movement H is equal to the sum of the average control delays (d_{15}, d_7) at each of the approach flows v_{15}, v_7 along its path. Furthermore, LOS F is defined to occur when either the v/c ratio or the average R_Q for any of the lane groups that contain this O-D exceeds 1, where R_Q refers to the average per lane queue storage ratio within the lane group.

Exhibit 22-12
Illustration of the LOS Concept at an Interchange with Roundabouts

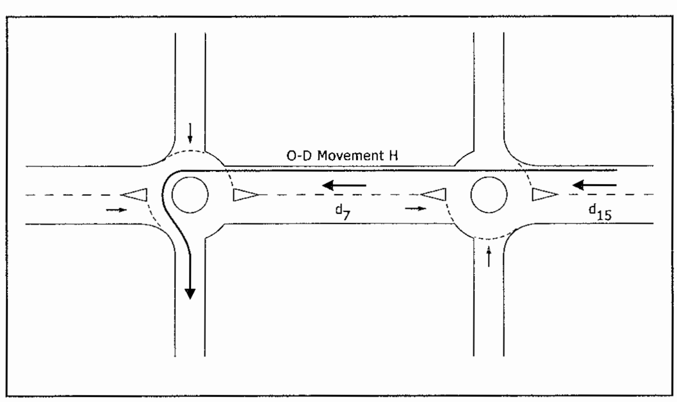


Exhibit 22-13 summarizes the LOS criteria for each O-D of an interchange with one or two roundabouts. The values presented in Exhibit 22-13 are greater than those for noninterchange roundabouts to reflect the fact that some of the O-D movements might travel through two roundabouts while others might travel through only one. The values are also generally lower than the respective ones for signalized interchanges, since drivers would likely expect lower delays at roundabouts.

Exhibit 22-13
LOS Criteria for Interchanges with Roundabouts

Control Delay (s/veh)	O-D LOS: $v/c < 1$ and $R_Q < 1$ for All Approaches	O-D LOS: $v/c > 1$ for Any Approach	O-D LOS: $R_Q > 1$ for Any Approach
≤15	A	F	F
>15–25	B	F	F
>25–35	C	F	F
>35–50	D	F	F
>50–75	E	F	F
>75	F	F	F

Other Interchange Types

Interchange types and control not explicitly included in this chapter (e.g., two-way STOP-controlled diamond interchanges) do not have LOS criteria defined on an O-D basis. In the absence of such LOS criteria, analyses of these interchange types and comparisons with other interchange types can be made by using control delay for each O-D and other applicable performance measures. These performance measures can be determined with procedures in this and other *Highway Capacity Manual* (HCM) chapters, alternative tools, or both, aggregated as appropriate into O-D performance measures by using the techniques in this chapter.

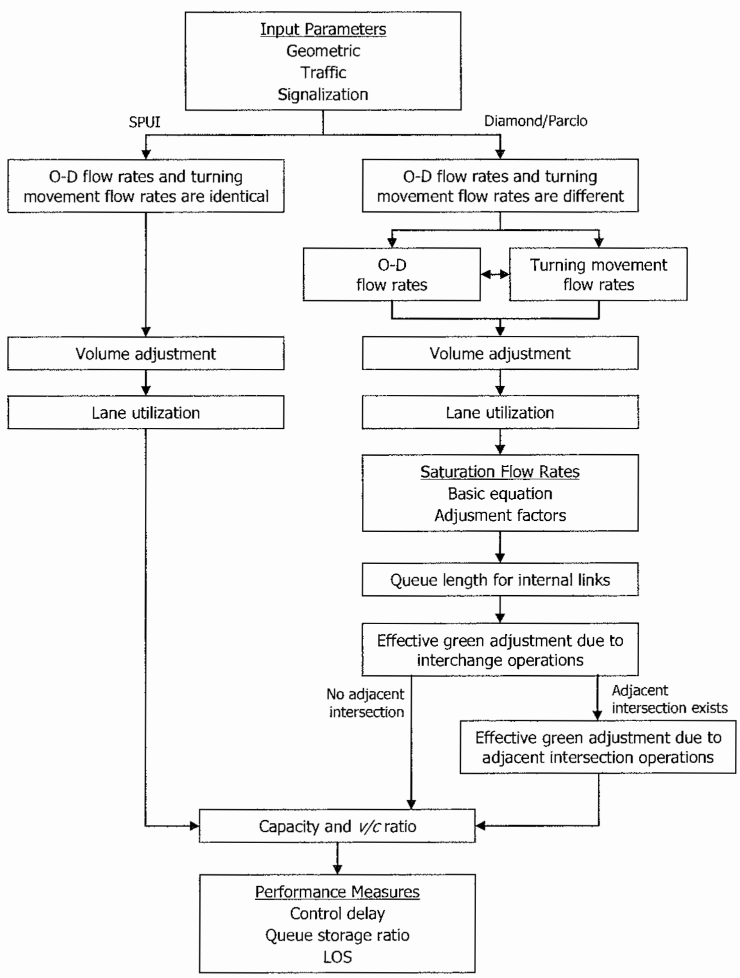
2. METHODOLOGIES

There are two general types of analysis for signalized interchange ramp terminals: (*a*) final design and traffic operational analysis and (*b*) operational analysis for interchange type selection. The methodology for final design and traffic operational analysis for signalized interchanges is presented first. The next section presents the methodology for analyzing interchanges with roundabouts and is followed by a brief discussion of operations at interchanges with unsignalized intersections. The last section presents the methodology used in interchange type selection.

FINAL DESIGN AND OPERATIONAL ANALYSIS FOR SIGNALIZED INTERCHANGES

Exhibit 22-14 summarizes the basic methodology for the final design and operational analysis of signalized interchange ramp terminals. The methodology is similar to that of Chapter 18, Signalized Intersections, with additional consideration for imbalanced lane utilizations, additional lost times due to downstream queues, demand starvation, and additional lost times due to interactions with closely spaced intersections.

Exhibit 22-14
Interchange Ramp Terminals Methodology: Final Design and Operational Analysis for Signalized Interchanges



The analysis of SPUIs is outlined on the left part of the graph. The graph highlights only the components added to the signalized intersection methodology for analyzing SPUIs. The right part of the graph highlights the components added to the signalized intersection methodology for analyzing diamond and parclo interchanges. Each of the steps outlined in Exhibit 22-14 is explained and discussed below.

The analysis begins with the assembly of all pertinent input data, such as geometric characteristics, traffic demands, and signalization information. Exhibit 22-15 provides a summary of all input data required to conduct an operational analysis for interchange ramp terminals.

Exhibit 22-15
Summary of Required Input Data for Final Design and Operational Analysis of Signalized Interchanges

Type of Condition	Parameter
Geometric conditions	Area type Number of lanes (N) Average lane width (W, ft) Grade (G, %) Existence of exclusive left- or right-turn lanes Length of storage for each lane group (L_a, ft) Distance corresponding to the internal storage between the two intersections in the interchange (D, ft) Distances corresponding to the internal storage between interchange intersections and adjacent closely spaced intersections (ft) Turning radii for all turning movements (ft)
Traffic conditions	Demand volume by O-D or turning movement (V, veh/h) Right-turn-on-red flow rates Base saturation flow rate (s_0, pc/hg/ln) Peak hour factor (PHF) Percent heavy vehicles (HV, %) Approach pedestrian flow rates (v_{ped}, ped/h) Approach bicycle flow rates (v_b, bicycles/h) Local bus stopping rate (N_B, buses/h) Parking activity (N_m, maneuvers/h) Arrival type (AT) Upstream filtering adjustment factor Approach speed (S_A, mi/h)
Signalization conditions	Type of signal control Phase sequence Cycle length (if appropriate) (C, s) Green times (if appropriate) (G, s) Yellow-plus-all-red change-and-clearance interval (intergreen) (Y, s) Offset (if appropriate) Maximum, minimum green, passage times, phase recall (for actuated control) Pedestrian push button Minimum pedestrian green (G_p, s) Phase plan

O-D Demands and Movement Demands

The analyst may have either O-D demands or intersection turning movements available for the study interchange. Since both are needed in the analysis, the first step in the methodology consists of calculating either the turning movements by using the O-D demands or the O-D demands by using the turning movements. If the interchange is a SPUI (i.e., only has one intersection), the O-D demands and the turning movement demands are the same, and the analysis proceeds similarly to the methodology of Chapter 18, Signalized Intersections, to estimate capacity, *v/c*, delay, and queue storage ratios. The

Applications section of this chapter provides guidance on converting O-D movements to turning movements and vice versa for each type of interchange configuration addressed in this methodology.

Lane Group Determination

As in the case of signalized interchanges, the methodology for interchange ramp terminals is disaggregate; that is, it is designed to consider individual intersection approaches and individual lane groups within approaches. The segmentation of the interchange into lane groups generally follows the same guidelines that apply for the analysis of signalized intersections.

Lane Utilization

Vehicles at interchanges do not distribute evenly among lanes in a lane group, and their lane selection is highly affected by their ultimate destination. For example, for two-intersection interchanges, when there is a high-volume left turn at the downstream intersection, traffic at the upstream intersection will gravitate toward the left lanes, while through and right-turning vehicles will tend toward the right. While this may occur at any intersection, because the internal link is generally short and because turning movements are typically high, there is generally greater variation in lane distribution and the adjustment factors that result from it. Segregation at the upstream intersection may occur by driver selection or by designated signing and striping.

To account for these phenomena, lane utilization models have been developed specifically for the external through approaches (surface streets) of two-intersection interchanges. The lane utilization factors for all other interchange approaches (freeway ramps, internal approaches, and SPUI approaches) are estimated by using the procedures of Chapter 18. These lane utilization factors are then used to adjust the saturation flow rates for each lane group.

Adjustment for Lane Utilization

The lane utilization factor accounts for the unequal distribution of traffic among the lanes in a lane group with more than one lane. The factor provides an adjustment to the base saturation flow rate. The adjustment factor is based on the flow in the lane with the highest volume and is calculated by Equation 22-2:

Equation 22-2

$$f_{LU} = \frac{1}{\%V_{Lmax} \times N}$$

where

f_{LU} = adjustment factor for lane utilization;

$\%V_{Lmax}$ = percent of the total approach flow in the lane with the highest volume, expressed as a decimal; and

N = number of lanes in lane group.

A series of models have been developed to predict $\%V_{Lmax}$ for the external arterial approaches of two-intersection interchanges as a function of the downstream turning movements. The remaining approaches should use lane utilization factors based on either field data or on values obtained from Exhibit 18-30.

Exhibit 22-16 through Exhibit 22-20 provide the models developed for each type of interchange configuration and for two-, three-, and four-lane arterials. In these exhibits, L1 represents the leftmost lane, L2 represents the second lane from the left, and so forth. The notation used in these exhibits is provided immediately after Exhibit 22-20.

The models estimate the percent of traffic expected to use each through lane as a function of the O-D demands in the subject approach. These models predict, for the external arterial approaches, the percent of traffic that is expected to use a particular lane as a function of the downstream turning movements. These turning movements are expressed in terms of their respective O-D flows. O-D flows (A through N) are shown in Exhibit 22-21 for each configuration type. When the patterns of the eastbound and westbound approaches are symmetrical, the calibration parameters for the eastbound and the westbound directions are identical, and only the O-D flows differ. Interchange approaches with identical turning movement patterns in the subject direction (eastbound or westbound) are grouped together, and the models developed apply to all configurations in the group. For example, the Parclo B-2Q, B-4Q, and AB-4Q–westbound approach are grouped together, and their lane utilization models are presented in Exhibit 22-16.

In Exhibit 22-16, Exhibit 22-18, and Exhibit 22-19, when an external approach has an exclusive right-turning lane, the O-D for that movement (v_F or v_G) should be assumed to be zero in the respective equation. In Exhibit 22-17, Exhibit 22-18, Exhibit 22-19, and Exhibit 22-20, when there is an additional leg in the upstream intersection, the analyst should use the lane utilization factors of Chapter 18. The models shown in these exhibits are valid for values of D less than 800 ft. The empirical models shown in Exhibit 22-16 through Exhibit 22-20 did not consider configurations with longer distances; for these longer distances between the two intersections vehicles tend not to preposition themselves in anticipation of a downstream turn. In those cases, and in the absence of field data, it is recommended that the default values of Exhibit 18-30 be used. In cases when the internal link contains dual left turns extending to the upstream approach, the volume in the most heavily traveled left-turning lane can be approximated as follows:

1. Use the model with number of lanes $N - 1$, where N is the number of lanes of the subject external approach;
2. Estimate the leftmost lane volume; and
3. Multiply by 0.515.

Research has shown that as operations approach congested conditions, the lane utilization factor tends to approach 1 (i.e., traffic becomes more uniformly distributed).

Actual lane volume distributions observed in the field should be used, if available, because these distributions are highly dependent on the existing land uses and access points in the vicinity of the interchange. A lane utilization factor of 1.0 can be used when uniform traffic distribution can be assumed across all lanes in the lane group or when a lane group comprises a single lane. The lane utilization factors are used in the next step of the methodology to adjust the saturation flow rates for each lane group of the interchange.

Exhibit 22-16
Lane Utilization Models for the External Arterial Approaches of Diamond Interchanges

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} - 0.154 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.187 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.0181 \times \left(\frac{D \times v_E}{10^6}\right)$
Eastbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{2} - 0.154 \times \left(\frac{v_G}{v_G + v_H + v_I}\right) + 0.187 \times \left(\frac{v_H}{v_G + v_H + v_I}\right) - 0.0181 \times \left(\frac{D \times v_H}{10^6}\right)$
Westbound Right lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} - 0.245 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.465 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Eastbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} + 0.609 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.326 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{3} - 0.245 \times \left(\frac{v_G}{v_G + v_H + v_I}\right) + 0.465 \times \left(\frac{v_H}{v_G + v_H + v_I}\right)$
Westbound Middle lane (% V_{L2})	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane (% V_{L3})	$\%V_{L3} = \frac{1}{3} + 0.609 \times \left(\frac{v_G}{v_G + v_H + v_I}\right) - 0.326 \times \left(\frac{v_H}{v_G + v_H + v_I}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} - 0.328 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) + 0.684 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Eastbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Eastbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} + 0.64 \times \left(\frac{v_F}{v_E + v_F + v_I}\right) - 0.233 \times \left(\frac{v_E}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1} = \frac{1}{4} - 0.328 \times \left(\frac{v_G}{v_G + v_H + v_I}\right) + 0.684 \times \left(\frac{v_H}{v_G + v_H + v_I}\right)$
Westbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane (% V_{L4})	$\%V_{L4} = \frac{1}{4} + 0.64 \times \left(\frac{v_G}{v_G + v_H + v_I}\right) - 0.233 \times \left(\frac{v_H}{v_G + v_H + v_I}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.
If there is an exclusive right-turn lane on the external approach, then the respective O-D demand (v_F or v_G) should be zero in the respective equation.

Exhibit 22-17
Lane Utilization Models for the External Arterial Approaches of Parclo A-2Q

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1}=\frac{1}{2}-0.527\times\left(\frac{v_E}{v_E+v_I}\right)$
Eastbound Right lane (% V_{L2})	$\%V_{L2}=1-\%V_{L1}$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1}=\frac{1}{2}-0.527\times\left(\frac{v_H}{v_H+v_J}\right)$
Westbound Right lane (% V_{L2})	$\%V_{L2}=1-\%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1}=\frac{1}{3}-0.363\times\left(\frac{v_E}{v_E+v_I}\right)$
Eastbound Middle lane (% V_{L2})	$\%V_{L2}=1-\%V_{L1}-\%V_{L3}$
Eastbound Right lane (% V_{L3})	$\%V_{L3}=\frac{1}{3}+0.655\times\left(\frac{v_E}{v_E+v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1}=\frac{1}{3}-0.363\times\left(\frac{v_H}{v_H+v_J}\right)$
Westbound Middle lane (% V_{L2})	$\%V_{L2}=1-\%V_{L1}-\%V_{L3}$
Westbound Right lane (% V_{L3})	$\%V_{L3}=\frac{1}{3}+0.655\times\left(\frac{v_H}{v_H+v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane (% V_{L1})	$\%V_{L1}=\frac{1}{4}-0.257\times\left(\frac{v_E}{v_E+v_I}\right)$
Eastbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2}=(1-\%V_{L1}-\%V_{L4})/2$ $\%V_{L3}=(1-\%V_{L1}-\%V_{L4})/2$
Eastbound Right lane (% V_{L4})	$\%V_{L4}=\frac{1}{4}+0.747\times\left(\frac{v_E}{v_E+v_I}\right)$
Westbound Leftmost lane (% V_{L1})	$\%V_{L1}=\frac{1}{4}-0.257\times\left(\frac{v_H}{v_H+v_J}\right)$
Westbound Middle lanes (% V_{L2}, % V_{L3})	$\%V_{L2}=(1-\%V_{L1}-\%V_{L4})/2$ $\%V_{L3}=(1-\%V_{L1}-\%V_{L4})/2$
Westbound Right lane (% V_{L4})	$\%V_{L4}=\frac{1}{4}+0.747\times\left(\frac{v_H}{v_H+v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.

If the intersection for which lane utilizations are being estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

Exhibit 22-18
Lane Utilization Models for the External Arterial Approaches of Parclo B-2Q, B-4Q, and AB-4Q (Westbound Only)

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{2} + 0.387 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.344 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Eastbound Right lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1}$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{2} + 0.387 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.344 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Right lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{3} + 0.559 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.218 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Eastbound Middle lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Eastbound Right lane ($\%V_{L3}$)	$\%V_{L3} = \frac{1}{3} - 0.429 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) + 0.695 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{3} + 0.559 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.218 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Middle lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane ($\%V_{L3}$)	$\%V_{L3} = \frac{1}{3} - 0.429 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) + 0.695 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{4} + 0.643 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) - 0.103 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Eastbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Eastbound Right lane ($\%V_{L4}$)	$\%V_{L4} = \frac{1}{4} - 0.359 \times \left(\frac{v_E}{v_E + v_F + v_I}\right) + 0.794 \times \left(\frac{v_F}{v_E + v_F + v_I}\right)$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{4} + 0.643 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) - 0.103 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$
Westbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane ($\%V_{L4}$)	$\%V_{L4} = \frac{1}{4} - 0.359 \times \left(\frac{v_H}{v_G + v_H + v_J}\right) + 0.794 \times \left(\frac{v_G}{v_G + v_H + v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.

If there is an exclusive right-turn lane on the external approach, then the respective O-D demand (v_F or v_G) should be zero in the respective equation.

If the intersection for which lane utilizations are being estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

Exhibit 22-19
Lane Utilization Models for the External Arterial Approaches of Parclo A-4Q, AB-2Q (Eastbound Only), and AB-4Q (Eastbound Only)

TWO LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1}=\frac{1}{2}-0.306\times\left(\frac{v_F}{v_E+v_F+v_I}\right)-0.484\times\left(\frac{v_E}{v_E+v_F+v_I}\right)$
Eastbound Right lane ($\%V_{L2}$)	$\%V_{L2}=1-\%V_{L1}$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1}=\frac{1}{2}-0.306\times\left(\frac{v_H}{v_G+v_H+v_J}\right)-0.484\times\left(\frac{v_G}{v_G+v_H+v_J}\right)$
Westbound Right lane ($\%V_{L2}$)	$\%V_{L2}=1-\%V_{L1}$
THREE LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1}=\frac{1}{3}-0.333\times\left(\frac{v_F}{v_E+v_F+v_I}\right)-0.289\times\left(\frac{v_E}{v_E+v_F+v_I}\right)$
Eastbound Middle lane ($\%V_{L2}$)	$\%V_{L2}=1-\%V_{L1}-\%V_{L3}$
Eastbound Right lane ($\%V_{L3}$)	$\%V_{L3}=\frac{1}{3}+0.579\times\left(\frac{v_F}{v_E+v_F+v_I}\right)+0.428\times\left(\frac{v_E}{v_E+v_F+v_I}\right)$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1}=\frac{1}{3}-0.333\times\left(\frac{v_H}{v_G+v_H+v_J}\right)-0.289\times\left(\frac{v_G}{v_G+v_H+v_J}\right)$
Westbound Middle lane ($\%V_{L2}$)	$\%V_{L2}=1-\%V_{L1}-\%V_{L3}$
Westbound Right lane ($\%V_{L3}$)	$\%V_{L3}=\frac{1}{3}+0.579\times\left(\frac{v_H}{v_G+v_H+v_J}\right)+0.428\times\left(\frac{v_G}{v_G+v_H+v_J}\right)$
FOUR LANES IN THE LANE GROUP	
Eastbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1}=\frac{1}{4}-0.233\times\left(\frac{v_F}{v_E+v_F+v_I}\right)-0.237\times\left(\frac{v_E}{v_E+v_F+v_I}\right)$
Eastbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2}=(1-\%V_{L1}-\%V_{L4})/2$ $\%V_{L3}=(1-\%V_{L1}-\%V_{L4})/2$
Eastbound Right lane ($\%V_{L4}$)	$\%V_{L4}=\frac{1}{4}+0.703\times\left(\frac{v_F}{v_E+v_F+v_I}\right)+0.641\times\left(\frac{v_E}{v_E+v_F+v_I}\right)$
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1}=\frac{1}{4}-0.233\times\left(\frac{v_H}{v_G+v_H+v_J}\right)-0.237\times\left(\frac{v_G}{v_G+v_H+v_J}\right)$
Westbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2}=(1-\%V_{L1}-\%V_{L4})/2$ $\%V_{L3}=(1-\%V_{L1}-\%V_{L4})/2$
Westbound Right lane ($\%V_{L4}$)	$\%V_{L4}=\frac{1}{4}+0.703\times\left(\frac{v_H}{v_G+v_H+v_J}\right)+0.641\times\left(\frac{v_G}{v_G+v_H+v_J}\right)$

Notes: Definitions of variables are provided after Exhibit 22-20.

If there is an exclusive right-turn lane on the external approach, then the respective O-D demand (v_F or v_G) should be zero in the respective equation.

If the intersection for which lane utilizations are being estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

Exhibit 22-20
Lane Utilization Models for the External Arterial Approaches of Parclo AB-2Q (Westbound Only) Interchanges

TWO LANES IN THE LANE GROUP	
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{2} + 0.468 \times \left(\frac{v_H}{v_H + v_J} \right)$
Westbound Right lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1}$
THREE LANES IN THE LANE GROUP	
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{3} + 0.735 \times \left(\frac{v_H}{v_H + v_J} \right)$
Westbound Middle lane ($\%V_{L2}$)	$\%V_{L2} = 1 - \%V_{L1} - \%V_{L3}$
Westbound Right lane ($\%V_{L3}$)	$\%V_{L3} = \frac{1}{3} - 0.308 \times \left(\frac{v_H}{v_H + v_J} \right)$
FOUR LANES IN THE LANE GROUP	
Westbound Leftmost lane ($\%V_{L1}$)	$\%V_{L1} = \frac{1}{4} + 0.768 \times \left(\frac{v_H}{v_H + v_J} \right)$
Westbound Middle lanes ($\%V_{L2}$, $\%V_{L3}$)	$\%V_{L2} = (1 - \%V_{L1} - \%V_{L4})/2$ $\%V_{L3} = (1 - \%V_{L1} - \%V_{L4})/2$
Westbound Right lane ($\%V_{L4}$)	$\%V_{L4} = \frac{1}{4} - 0.202 \times \left(\frac{v_H}{v_H + v_J} \right)$

Note: Definitions of variables are provided after Exhibit 22-20.
If the intersection where lane utilizations are estimated has an additional leg, the analyst should not use the equations of this exhibit. The procedures of Chapter 18 should be used instead.

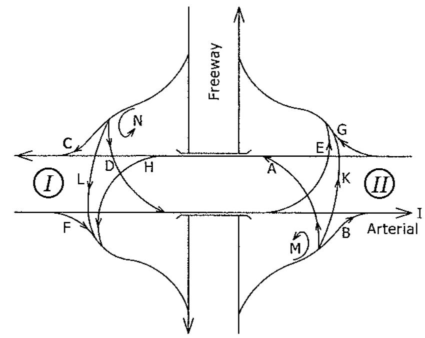
Definitions of variables used in Exhibit 22-16 through Exhibit 22-20 are as follows:

$\%V_{Li}$ = percent of traffic present in lane L*i*, where L1 represents the leftmost lane, L2 represents the second lane from the left, and so forth;

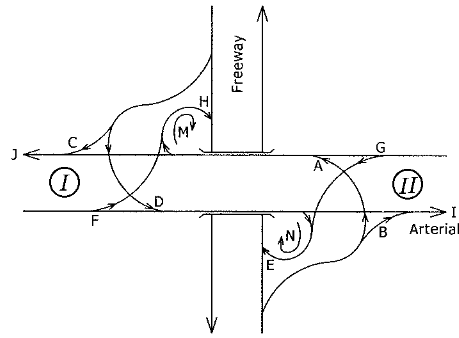
D = distance between the two intersections of the interchange (ft); and

v_i = O-D demand flow rates for movement *i* (veh/h), as illustrated in Exhibit 22-21 for each interchange type.

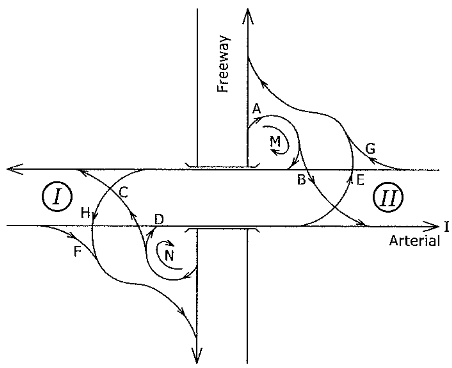
Exhibit 22-21
O-D Flows for Each Interchange Configuration



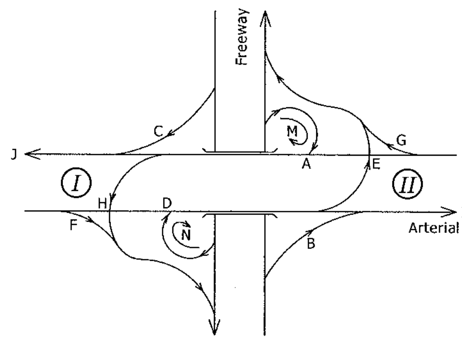
(a) Diamond Interchange



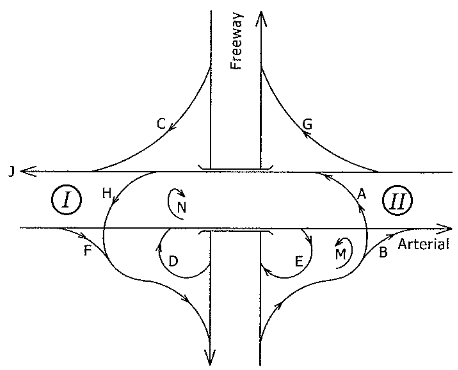
(b) Parclo A-2Q



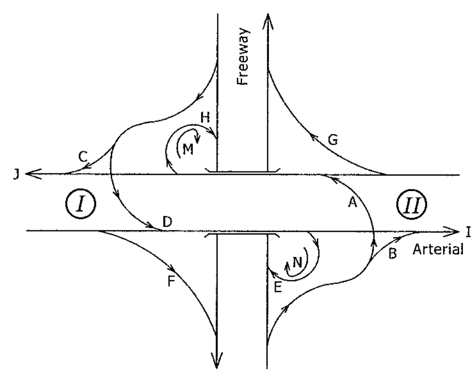
(c) Parclo B-2Q



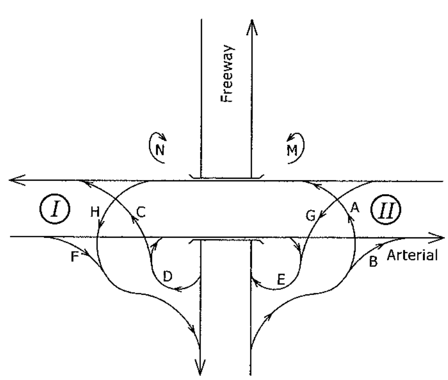
(d) Parclo B-4Q



(e) Parclo AB-4Q



(f) Parclo A-4Q



(g) Parclo AB-2Q

Saturation Flow Rates

The saturation flow rate for each lane group can either be measured in the field or estimated with the following equation:

Equation 22-3

$$s = s_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{RT} f_{LT} f_{Lpb} f_{Rpb} f_{LU} f_v$$

where

s = adjusted saturation flow rate (veh/h),

s_0 = base saturation flow rate per lane (1,900 pc/hg/ln),

N = number of lanes in the lane group,

f_w = adjustment factor for lane width (from Chapter 18),

f_{HV} = adjustment factor for heavy-vehicle presence (from Chapter 18),

f_g = adjustment factor for approach grade (from Chapter 18),

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to the lane group (from Chapter 18),

f_{bb} = adjustment factor for local bus blockage (from Chapter 18),

f_a = adjustment factor for the area type (from Chapter 18),

f_{RT} = adjustment for right-turning vehicle presence in the lane group (from Chapter 18),

f_{LT} = adjustment for left-turning vehicle presence in the lane group (from Chapter 18),

f_{Lpb} = pedestrian adjustment factor for left turns (from Chapter 18),

f_{Rpb} = pedestrian–bicycle adjustment factor for right turns (from Chapter 18),

f_{LU} = adjustment factor for lane utilization (from Equation 22-2 and Exhibit 22-16 to Exhibit 22-20 as discussed in the previous section), and

f_v = adjustment for traffic pressure (from Exhibit 22-22).

All factors in Equation 22-3 are obtained from Chapter 18, Signalized Intersections, except the last two. A third adjustment factor, f_R, which quantifies the effect of turn radius on the saturation flow rate of a left- or right-turn movement, is used to modify the adjustment factors for protected turn movements provided in Chapter 18. The lane utilization factor was discussed in the previous section, while the other two are discussed below.

Adjustment for Traffic Pressure

Saturation flow rates have generally been found to be higher during peak traffic demand periods than during off-peak periods (3). Traffic pressure reflects the display of aggressive driving behavior for a large number of drivers during high-demand traffic conditions. Under such conditions, a large number of drivers accept shorter headways during queue discharge than they would under different circumstances.

The effect of traffic pressure has been found to vary by traffic movement. The left-turn movements tend to be more affected by traffic pressure than through or right movements. To account for this phenomenon, the saturation flow rates at interchange approaches are adjusted by using the traffic pressure factor. This factor is computed with the following equation:

$$f_v = \begin{cases} \dfrac{1}{1.07 - 0.00672 v_i'} & \text{(left turn)} \\ \dfrac{1}{1.07 - 0.00486 v_i'} & \text{(through or right turn)} \end{cases}$$

Equation 22-4

where f_v is the adjustment factor for traffic pressure and v_i' is demand flow rate per cycle per lane (veh/cycle/ln).

For values of v_i' higher than 30 veh/cycle/ln, 30 veh/cycle/ln should be used, since the effects of demands higher than that value are not known.

Exhibit 22-22 tabulates the results of Equation 22-4 for various demands and for each turning movement type.

Exhibit 22-22
Adjustment Factor for Traffic Pressure (f_v)

Demand Flow Rate v_i' (veh/cycle/ln)	Movement Type: Left Turn	Movement Type: Through and Right Turn
3.0	0.953	0.947
6.0	0.971	0.961
9.0	0.991	0.974
12.0	1.011	0.988
15.0	1.032	1.003
18.0	1.054	1.018
21.0	1.077	1.033
24.0	1.100	1.049

When the lane group is shared by several movements, the adjustment factor for traffic pressure is estimated as the weighted (on the basis of the respective flows) average of the respective movements.

Adjustment for Turn Radius Effects on Left- or Right-Turning Movements

Traffic movements that discharge along a curved travel path do so at rates lower than those of through movements (3). The turning radius has been found to have a unique effect on saturation flows for turning movements at interchanges (3). The adjustment factor to account for the effects of travel path radius, f_R, is calculated with the following equation:

$$f_R = \frac{1}{1 + \dfrac{5.61}{R}}$$

Equation 22-5

where R is the radius of curvature of the left- or right-turning path (at the center of the path), in feet.

The turning radius adjustment factor f_R is used to revise the adjustment factors for protected turn movements provided in Chapter 18. The revised left- and right-turn adjustment factors are as follows:

For protected, exclusive left-turn lanes:

Equation 22-6

$$f_{LT} = f_R$$

For protected, shared left-turn lanes:

Equation 22-7

$$f_{LT} = \frac{1}{1 + P_{LT}\left(\frac{1}{f_R} - 1\right)}$$

For protected, exclusive right-turn lanes:

Equation 22-8

$$f_{RT} = f_R$$

For protected, shared right-turn lanes:

Equation 22-9

$$f_{RT} = \frac{1}{1 + P_{RT}\left(\frac{1}{f_R} - 1\right)}$$

Exhibit 22-23 tabulates the adjustment factor for turn radius for several radii.

Exhibit 22-23
Adjustment Factor for Turn Radius (f_R)

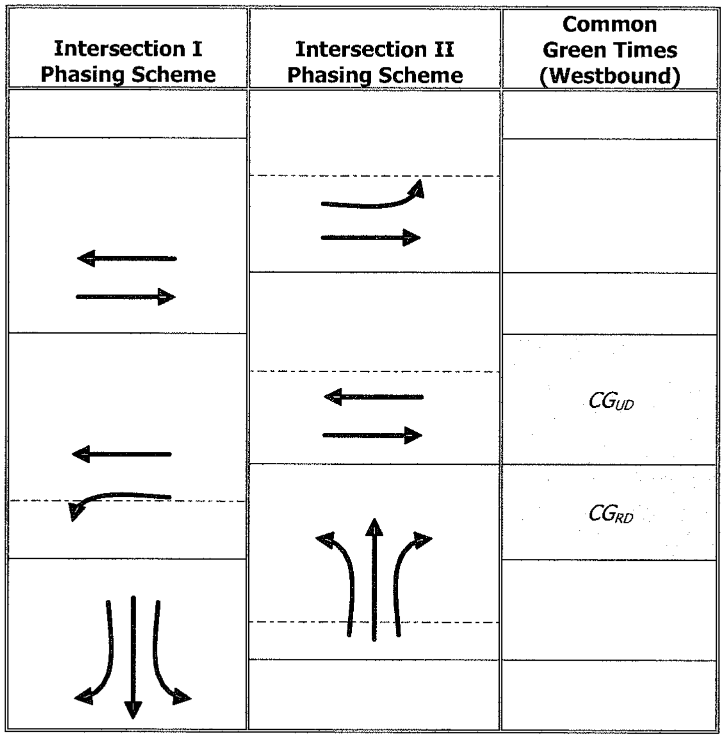
Radius of the Travel Path (ft)	Movement Type	
	Left and Right Turn	Through
25	0.817	1.00
50	0.899	1.00
100	0.947	1.00
150	0.964	1.00
200	0.973	1.00
250	0.978	1.00
300	0.982	1.00
350	0.984	1.00

When the lane group is shared by several movements, the adjustment factor for turn radii is estimated as the weighted (on the basis of the respective flows) average of the respective movements. The adjustment factors for permissive phasing are estimated by using the procedures of Chapter 18.

Additional Lost Time for the Upstream Approaches due to the Presence of a Downstream (Internal) Link Queue

The presence of a downstream queue may reduce or block the discharge of the upstream movements, increasing the amount of lost time for the upstream phases. In the analysis of interchange ramp terminals, the effects of the presence of a queue at the downstream link (through movement) are considered by estimating the amount of additional lost time experienced at the upstream intersection. The methodology takes into consideration the duration of common green times between various phases at the two intersections. In this chapter common green time between Phase A in Intersection I and Phase B in Intersection II is defined as the amount of time (in seconds) during which both phases have a green indication. Exhibit 22-24 provides an illustrative example of common green times between the upstream and downstream through phases (CG_{UD}) and between the upstream ramp and the downstream through phases (CG_{RD}).

Exhibit 22-24
Illustration of Common Green Times



The additional lost time due to the presence of a downstream queue in the internal through movement is calculated for each of the upstream approaches with the following equations:

Additional lost time on the external arterial approach:

$$L_{D\text{-}A} = G_A - 0.106 \times DQ_A - 5.39 \times \frac{CG_{UD}}{C}$$

Equation 22-10

Additional lost time on the ramp approach:

$$L_{D\text{-}R} = G_R - 0.106 \times DQ_R - 5.39 \times \frac{CG_{RD}}{C}$$

Equation 22-11

where

$L_{D\text{-}A}$ = lost time on external arterial approach due to presence of downstream queue (s),

$L_{D\text{-}R}$ = lost time on external ramp approach due to presence of downstream queue (s),

G_A = green interval for external arterial approach (s),

G_R = green interval for left-turning ramp movement (s),

DQ_A = distance to downstream queue at beginning of upstream arterial green (ft),

DQ_R = distance to downstream queue at beginning of upstream ramp green (ft),

CG_{UD} = common green time between upstream and downstream arterial through green (s),

CG_{RD} = common green time between upstream ramp green and downstream arterial through green (s), and

C = cycle length (s).

If Equation 22-10 or Equation 22-11 results in negative values, then the respective lost times $L_{D\text{-}A}$ or $L_{D\text{-}R}$ are zero. Furthermore, if DQ_A or DQ_R exceeds 200 ft, then the lost time will be zero.

DQ_A and DQ_R are calculated as follows:

Equation 22-12

$$DQ_A = D - Q_A$$

Equation 22-13

$$DQ_R = D - Q_R$$

where

D = distance corresponding to storage space between the two intersections of the interchange (ft),

Q_A = estimated average per lane queue length for through movement in downstream (internal) link at beginning of upstream arterial Phase A (ft), and

Q_R = estimated average per lane queue length for through movement in downstream (internal) link at beginning of upstream ramp Phase R (ft).

The downstream queue length (averaged across all through lanes) at the beginning of each upstream phase is calculated with the following equations:

Queue at the beginning of the upstream arterial Phase A:

Equation 22-14

$$Q_A = \left[0.0107\frac{v_R}{N_R} - 7.96 \times \frac{G_D}{C} - 0.082CG_{UD} + 7.96\frac{G_R}{C}\right] \times L_h$$

Queue at the beginning of the upstream ramp Phase R:

Equation 22-15

$$Q_R = \left[0.0107\frac{v_A}{N_A} - 7.96 \times \frac{G_D}{C} - 0.082CG_{RD} + 7.96\frac{G_A}{C}\right] \times L_h$$

where

Q_A = queue at the beginning of upstream arterial Phase A (ft);

Q_R = queue at the beginning of upstream ramp Phase R (ft);

v_R = ramp flow feeding subject queue (veh/h);

v_A = arterial flow feeding subject queue (veh/h);

N_R = number of ramp lanes feeding the subject queue;

N_A = number of arterial lanes feeding the subject queue;

G_R = green interval for upstream left ramp movement (s);

G_A = green interval for upstream arterial through movement (s);

G_D = green interval for downstream arterial through movement (s);

CG_{UD} = common green time between upstream arterial green and downstream through green (s);

CG_{RD} = common green time between upstream ramp green and downstream through green (s);

L_h = average queue spacing in a stationary queue, measured from front bumper to front bumper between successive vehicles (ft/veh); and

C = cycle length (s).

The variables v_R, N_R, v_A, and N_A refer to the movement flows that feed the subject queue. For example, for the diamond interchange, v_R is the left-turning flow from the ramp, and the variable becomes $v_{\text{Ramp-}L}$. For actuated signals, the analyst should first determine the equivalent pretimed signal timing plan on the basis of the average duration of each phase during the study hour and estimate the parameters described above on the basis of that plan.

If Q_A or Q_R is calculated to be less than zero, then the expected queue is zero, and no additional lost time due to the presence of a downstream queue will be experienced. Similarly, if the lost times $L_{D\text{-}A}$ or $L_{D\text{-}R}$ are estimated to be negative, then the expected lost time will be zero for the respective approach. Conversely, if Q_A or Q_R exceeds the available storage, its value should be set equal to that storage, and the respective distance to the downstream queue, DQ_A or DQ_R, should be set to zero.

Additional Lost Time for the Downstream (Internal) Approaches due to Demand Starvation

This methodology accounts for the effects of demand starvation in interchange operations by computing the lost time experienced at the downstream intersection that results from demand starvation. Lost time due to demand starvation (L_{DS}) is defined as the amount of green time during which there is no queue present to be discharged from the internal link *and* there are no arrivals from either of the upstream approaches due to signalization. The common green time between two phases that may lead to demand starvation is called *common green time with demand starvation potential* (CG_{DS}). Exhibit 22-25 provides an illustrative example of an interval with demand starvation potential. In that example, there is potential for demand starvation for the westbound internal through movement of the interchange.

The following equation is used to estimate lost time due to demand starvation:

$$L_{DS} = CG_{DS} - Q_{\text{INITIAL}} \times h_I$$

Equation 22-16

where

L_{DS} = additional lost time due to demand starvation (s);

CG_{DS} = common green time with demand starvation potential (s);

h_I = saturation headway for internal through approach (= 3,600/saturation flow per lane) (s); and

Q_{INITIAL} = length of queue stored at internal approach at beginning of interval during which this approach has demand starvation potential, calculated from Equation 22-17.

Equation 22-17

$$Q_{\text{INITIAL}} = \left(\frac{v_{\text{Ramp-}L} \times C}{N_{\text{Ramp-}L} \times 3{,}600} - \frac{(CG_{RD} - t_L)}{h_I} \right) + \left(\frac{v_{\text{Arterial}} \times C}{N_{\text{Arterial}} \times 3{,}600} - \frac{(CG_{UD} - t_L)}{h_I} \right)$$

where

$v_{\text{Ramp-}L}$ = upstream ramp left-turning flow (v/h),

v_{Arterial} = upstream arterial through flow (v/h),

C = cycle length (s),

$N_{\text{Ramp-}L}$ = number of lanes for upstream ramp left-turning movement,

N_{Arterial} = number of lanes for upstream arterial through movement,

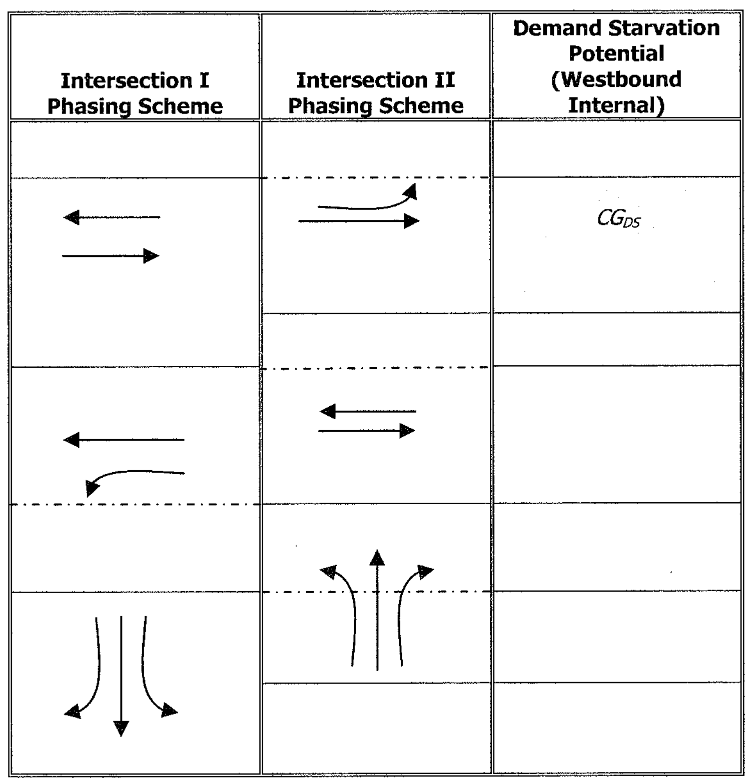
CG_{RD} = common green time between upstream ramp and downstream through green phase (s),

CG_{UD} = common green time between upstream through and downstream through green phase (s),

h_I = saturation headway for internal through approach (= 3,600/saturation flow per lane) (s), and

t_L = lost time per phase (s).

Exhibit 22-25
Illustration of Interval with Demand Starvation Potential



Equation 22-16 calculates the amount of time that would not be used because the internal link queue has completely discharged and the upstream demand is blocked and cannot arrive to the internal link stop line. The initial queue at the beginning of the demand starvation interval is estimated as a function of the demands of the upstream approaches and of the respective common intervals between the upstream and downstream green.

Equation 22-17 is valid for values of CG_{RD} and $CG_{UD} \geq t_L$. If CG_{RD} or $CG_{UD} < t_L$, the analyst should assume that CG_{RD} or $CG_{UD} = t_L$. Also, in applying Equation 22-17 it is assumed that no vehicles will have to wait for more than one cycle (i.e., none of the approaches is oversaturated). If the time required to discharge the queue is equal to or larger than the CG_{DS}, the lost time due to demand starvation will be zero. The model for estimating lost time due to demand starvation assumes uniform arrivals and departures and assumes that operations at the interchange are not oversaturated.

Effective Green Adjustments

The effective green adjustment involves two components: (*a*) adjustment in the effective green of the upstream approaches due to the presence of a downstream queue and (*b*) adjustment in the effective green of the downstream (internal) approaches due to demand starvation.

The adjusted lost time t_L' for the arterial approaches and for the ramp approaches is estimated as follows:

Arterial approaches:

$$t_L' = l_1 + L_{D\text{-}A} + Y - e$$

Equation 22-18

Ramp approaches:

$$t_L' = l_1 + L_{D\text{-}R} + Y - e$$

Equation 22-19

where

t_L' = adjusted lost time (i.e., time when the signalized intersection is not used effectively by any movement) (s),

l_1 = start-up lost time (s),

$L_{D\text{-}A}$ = lost time on external arterial approach due to presence of a downstream queue (s),

$L_{D\text{-}R}$ = lost time on external ramp approach due to presence of a downstream queue (s),

Y = yellow-plus-all-red change-and-clearance interval (s), and

e = extension of effective green time into the clearance interval (s).

The adjusted lost time t_L'' for the internal approaches is estimated as follows:

$$t_L'' = l_1 + L_{DS} + Y - e$$

Equation 22-20

where L_{DS} is the additional lost time due to demand starvation (s).

The effective green time adjusted due to the presence of a downstream queue is then calculated for the external approaches by using the following equation:

$$g' = G + Y - t_L'$$

Equation 22-21

where g' is the effective green time adjusted due to presence of a downstream queue (s), G is the green time (s), and t_L' is the adjusted lost time for external approaches (s).

Similarly, the effective green time adjusted due to demand starvation is calculated for the internal approaches as follows:

Equation 22-22

$$g' = G + Y - t''_L$$

where g' is the effective green time adjusted due to demand starvation (s), G is the green time (s), and t''_L is the adjusted lost time for the internal approaches (s).

Closely Spaced Adjacent Intersections

The presence of closely spaced signalized intersections in the vicinity of an interchange may affect the operations of the entire interchange system and can present unique operational challenges. First, the lane utilizations of the arterial approaches would be affected by the presence of the interchange as vehicles position themselves to make a turn downstream or as they enter the arterial from the interchange. Queuing from the adjacent intersections could affect the discharge rate of the upstream (internal) link of the interchange. Furthermore, demand starvation in the internal link can coexist with queues upstream, in the external approaches of the interchange. If this external link is short, queue spillback may affect the adjacent intersections and have a significant and long-lasting impact throughout the interchange area. Generally, closely spaced signalized intersections whose traffic signals are poorly timed can cause flow blockages on the next upstream link due to queue spillback, even during nominally undersaturated conditions.

In the analysis of interchange ramp terminals, the effects of the presence of closely spaced intersections are considered by adjusting the lane utilizations of the intersections' arterial approaches, by estimating the amount of additional lost time experienced at the upstream intersection due to the presence of the downstream queue, and by estimating the additional lost time due to demand starvation.

The lane utilization factors for the through approaches of closely spaced intersections should be estimated by subtracting 0.05 from the lane utilization factors obtained from Exhibit 18-30. Research has shown that those lane utilization factors are generally lower than those at a typical intersection approach.

The additional lost times experienced at the approaches to closely spaced intersections are estimated as discussed in the previous section. A brief overview is provided here for convenience.

Additional lost time may be experienced at any of the upstream approaches to the closely spaced intersections. The additional lost time due to the presence of the downstream queue is calculated for each of the upstream approaches i by using the following equation:

Equation 22-23

$$L_{D\text{-}U_i} = G_{U_i} - 0.106 \times DQ_i - 5.39 \times \frac{CG_{U_iD}}{C}$$

where

$L_{D\text{-}U_i}$ = lost time on upstream approach i due to presence of a downstream queue (s),

G_{U_i} = green interval for upstream approach i (s),

DQ_i = distance to downstream queue at beginning of upstream green for approach i (ft), and

CG_{U_iD} = common green time between upstream approach i and downstream through green (s).

The distance to the downstream queue at the beginning of the upstream green is calculated on the basis of the estimated average per lane queue length (in feet) for the through movement in the downstream link at the beginning of the respective upstream phase with Equation 22-12 through Equation 22-15.

When a significant portion of the traffic demand is held at the upstream adjacent intersection, demand starvation can occur on the external approaches to the interchange. The lost time caused by demand starvation on the external approaches to the interchange is estimated with the following equation:

$$L_{DS} = CG_{DS} - Q_{\mathrm{INITIAL}} \times h_I$$

Equation 22-24

where

L_{DS} = additional lost time due to demand starvation (s),

CG_{DS} = common green time with demand starvation potential (s),

h_I = saturation headway for through approach (= 3,600/saturation flow per lane) (s), and

Q_{INITIAL} = initial queue length at beginning of interval with demand starvation potential.

When the operations of adjacent closely spaced intersections affect and are affected by operations at the interchange, the external and internal approaches of the interchange could experience both lost time due to a downstream queue and demand starvation. For example, the internal approach of a diamond interchange may experience lost time due to a downstream queue created at the downstream intersection, and at the same time it may experience demand starvation. In those cases, the procedures of this chapter should not be applied; simulation or other alternative tools should be used instead.

LOS Determination

The determination of LOS for each O-D in the interchange involves the calculation of three performance measures: the queue storage ratios (R_Q), the v/c ratios, and the average control delays. First, the queue storage ratios and v/c ratios for each lane group are estimated. If for any given lane group one or both of these variables exceed 1.0, then the LOS for every O-D that travels through this particular lane group will be F. Next, the average control delay for each lane group is estimated. Finally, the average control delay for each O-D is estimated as the sum of the control delays for each lane group through which the O-D travels.

Queue Storage Ratio Estimation

The procedure to estimate the queue storage ratio (R_Q) is described in detail in Chapter 31, Signalized Intersections: Supplemental.

v/c Ratio Estimation

For a given lane group i, X_i is computed with the following equation:

Equation 22-25

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i}$$

where

X_i = $(v/c)_i$ ratio for lane group i,

v_i = actual or projected demand flow rate for lane group i (veh/h),

s_i = saturation flow rate for lane group i (veh/h),

g_i = effective green time for lane group i (s), and

C = cycle length (s).

Note that the effective green time g should be replaced by the adjusted green time g' if there is additional lost time due to a downstream queue and by the adjusted green time g'' if there is lost time due to demand starvation.

Average Control Delay Estimation for Each O-D

The average control delay for each lane group and movement is estimated by using the procedures provided in Chapter 18, Signalized Intersections. The average control delay for each O-D is estimated as the total delay experienced by that O-D. If the O-D travels only through one intersection, then its average control delay is equal to the average control delay of the respective lane group. If the O-D travels through both intersections, then its average control delay is the sum of the delays experienced at each of the lane groups along its path.

Operations at the closely spaced intersections are generally assessed by using the procedures of Chapter 18. The additional lost time estimation, which is computed with the procedures of this chapter, is used to determine the adjusted effective green time for all affected approaches.

FINAL DESIGN AND OPERATIONAL ANALYSIS FOR INTERCHANGES WITH ROUNDABOUTS

Roundabouts are generally analyzed with the procedures of Chapter 21 of the HCM. This chapter provides guidance for translating O-D demands into movement demands at a roundabout to apply the procedures of Chapter 21.

Exhibit 22-26 defines the movements traveling through an interchange with two roundabouts, while Exhibit 22-27 lists the O-D demands contributing to each of these movements. For example, for diamond interchanges, O-D Movements G, H, and J constitute Movement 15 in Exhibit 22-26.

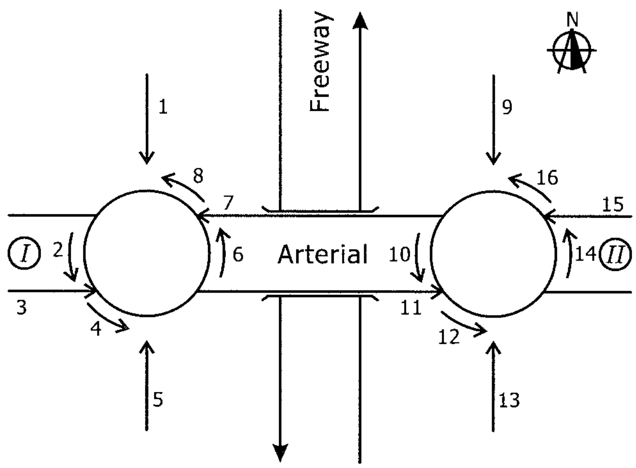


Exhibit 22-26
Illustration and Notation of O-D Demands at an Interchange with Roundabouts

Exhibit 22-27
Notation of O-D Demands at Interchanges with Roundabouts

Movement	Diamond	Parclo A-2Q	Parclo B-2Q	Parclo B-4Q
1	C, D, L, N	C, D, N	--	C
2	D, H, L, M, N	D, N	H, M, N	H, M
3	E, F, I	E, F	E, F, I	E, F, I
4	D, E, F, H, I, L, M, N	D, E, F, I, N	E, F, H, I, M	E, F, H, I, M
5	--	--	C	D, N
6	--	F	C	--
7	A, H, J, M	A, H, J, M	A, H, J, M	A, H, J, M
8	J, M	A, F, H, J, M	A, C, H, J, M	A, H, J, M
9	--	--	A, B, M	A, M
10	--	G	B	-
11	D, E, I, N	D, E, I, N	D, E, I, N	D, E, I, N
12	D, E, I, N	D, E, G, I, N	B, D, E	D, E, I, N
13	A, B, K, M	A, B, M	--	B
14	A, E, K, M, N	A, M	E, N	E, N
15	G, H, J	G, H, J	G, H, J	G, H, J
16	A, E, G, H, J, K, M, N	A, G, H, J, M	E, G, H, J, N	E, G, H, J, N
Movement	**SPUI**	**Parclo AB-4Q**	**Parclo A-4Q**	**Parclo AB-2Q**
1	C, D, L, N	C	C, D, N	--
2	D, H, L, M, N	H, M	D, N	H, M
3	E, F, I	E, F, I	E, F, I	E, F, I
4	D, E, I, N	E, F, H, I, M	D, E, F, I, N	E, F, H, I, M
5	A, B, K, M	D, N	--	C, D, N
6	A, E, K, M, N	--	--	C
7	G, H, J	A, H, J, M	A, H, J, M	A, H, J, M
8	A, H, J, M	A, H, J, M	A, H, J, M	A, C, H, J, M
9	--	--	--	--
10	--	--	--	G
11	--	D, E, I, N	D, E, I, N	D, E, I, N
12	--	D, E, I, N	D, E, I, N	D, E, G, I, N
13	--	A, B, M	A, B, M	A, B, M
14	--	A, M	A, M	A, M
15	--	G, H, J	G, H, J	G, H, J
16	--	A, G, H, J, M	A, G, H, J, M	A, G, H, J, M

Note: -- indicates movements that do not exist for a given interchange form.

In analyzing interchanges with roundabouts, Exhibit 22-26 and Exhibit 22-27 should be used to establish the roundabout movements. The procedures of Chapter 21 should then be applied to estimate the capacity and delay for each roundabout approach. Finally, Exhibit 22-13 should be used to determine the LOS for each O-D demand through the interchange.

INTERCHANGES WITH UNSIGNALIZED INTERSECTIONS

Interchanges with unsignalized intersections cannot be evaluated with the procedures of this chapter, since research has not yet been done on the operation of two closely spaced unsignalized intersections. In the absence of research, the intersections of such interchanges can be analyzed individually with the procedures of Chapter 19, Two-Way STOP-Controlled Intersections, or Chapter 20, All-Way STOP-Controlled Intersections.

OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

The operational analysis for interchange type selection can be used to evaluate the operational performance of various interchange types. It allows the user to compare eight fundamental types of interchanges for a given set of demand flows. The eight signalized interchange types covered by the interchange type selection analysis methodology are as follows:

1. SPUI,
2. Tight urban diamond interchange (TUDI),
3. Compressed urban diamond interchange (CUDI),
4. Conventional diamond interchange (CDI),
5. Parclo A—four quadrants (Parclo A-4Q),
6. Parclo A—two quadrants (Parclo A-2Q),
7. Parclo B—four quadrants (Parclo B-4Q), and
8. Parclo B—two quadrants (Parclo B-2Q).

Other types of signalized interchanges cannot be investigated with this interchange type selection analysis methodology. Also, the operational analysis methodology does not distinguish between the TUDI, CUDI, and CDI types. In general, the interchange type selection analysis methodology categorizes diamond interchanges by the distance between the centerlines of the ramp roadways that form the signalized intersections. This distance is generally between 200 and 400 ft for the TUDI, between 600 and 800 ft for the CUDI, and between 1,000 and 1,200 ft for the CDI.

The method is based on research (*4*). The research also provides a methodology for selecting unsignalized interchanges. Since unsignalized interchanges are not covered by this chapter, users should consult the original source for this information.

The methodology is based on the estimation of the sums of critical flow ratios through the interchange and their use to estimate interchange delay. A combination of simulation and field data was used to develop critical relationships for the methodology.

The sum of critical flow ratios is based on an identification of all flows served during a particular signal phase and the determination of maximum flow ratios among the movements served by that phase. The models are similar to those used in Chapter 18 for signalized intersections; they are modified to take into account the fact that each signal phase involves two signalized intersections. Interchange delay is defined as the total of all control delays experienced by all interchange movements involved in signalized ramp terminal movements divided by the sum of all external movement flows. Additional information is available in the source report (4).

Because signalization is not specified for an interchange type selection analysis, it is assumed that the following interchange types are operated by a single signal controller: SPUI, TUDI, and CUDI. All other types are assumed to be operated by separate controllers at each signalized ramp terminal. In all cases, optimal signal timing and phasing are assumed.

Inputs and Applications

This interchange type selection analysis methodology can be used in several ways:

1. For a given set of O-D interchange movements, eight basic types of signalized interchanges may be compared on the basis of interchange delay;
2. For a given type of interchange, the impact of intersection spacing on interchange delay can be examined (within the range of applicability for each interchange type); and
3. For a given type of interchange, the impact of the number of lanes on ramp and surface arterial approaches and the movements assigned to these lanes can be examined, again by using interchange delay as the measure of effectiveness.

For any of these applications, all interchange O-D movements must be specified, generally by using full peak-hour volumes. The interchange type selection methodology is not detailed enough to use flow rates or to consider such factors as the presence of heavy vehicles.

In addition, for any given computation, it is necessary to specify the number of lanes assigned to each phase movement and to specify the distance between the centerlines of the two ramps, measured along the surface arterial.

Saturation Flow Rates

Implementation of the interchange type selection methodology requires the adoption of default values for saturation flow rate. Research (4) suggests the use of 1,900 veh/hg/ln for some basic cases. This is, however, based on a suggested base saturation flow rate of 2,000 pc/hg/ln, which is higher than the default values suggested in Chapter 18, Signalized Intersections. For consistency with the base saturation flow rate of 1,900 pc/hg/ln specified in Chapter 18, and to recognize the impact of various movements on saturation flow rate, it is recommended that the default values shown in Exhibit 22-28 be used in conjunction with the interchange type selection methodology. Alternatively, and if relevant information is available, the default values provided in Chapter 18, Signalized Intersections (Exhibit 18-28), may be used.

Exhibit 22-28
Default Values of Saturation Flow Rate for Use with the Operational Analysis for Interchange Type Selection

Interchange Type	Default Saturation Flow Rate (veh/hg/ln) Left Turns	Through	Right Turns
SPUI	1,800	1,800	1,800
TUDI	1,700	1,800	1,800
CUDI	1,700	1,800	1,800
CDI	1,700	1,800	1,800
Parclo A-4Q	1,700	1,800	1,800
Parclo A-2Q	1,700	1,800	1,800
Parclo B-4Q	1,700	1,800	1,800
Parclo B-2Q	1,700	1,800	1,800

Where turning movements are in shared lanes, the "through" saturation flow rates should be used for analysis.

Step 1: Mapping O-D Flows into Interchange Movements

Since the primary objective of an interchange type selection analysis is to compare up to eight interchange types against a given set of design volumes, it is necessary first to convert a given set of design origin and destination volumes to movement flows through the signalized interchange. The methodology identifies volumes by signal phase by using the standard National Electrical Manufacturers Association (NEMA) numbering sequence for interchange phasing. Thus, movements are numbered 1 through 8 on the basis of the signal phase that accommodates the movement. Not all configurations and signalizations include all eight NEMA phases, and for some interchange forms some movements are not signalized and do not, therefore, contribute to interchange delay.

As for the operational analysis methodology, to simplify the mapping process, it is assumed that the freeway is oriented north-south and the surface arterial east-west. If the freeway is oriented in the east-west direction, rotate the interchange drawing or diagram clockwise until the freeway is in the north-south direction. In rotating clockwise, the westbound freeway direction becomes northbound and the eastbound freeway direction becomes southbound; the northbound arterial direction becomes eastbound and the southbound arterial direction becomes westbound. The methodology allows for separate consideration of freeway U-turn movements through the interchange. Thus, there are 14 basic movements that must be mapped for each interchange type.

For interchange types using two controllers, phase movements through the left (Intersection I) and right (Intersection II) intersections of the interchange are separately mapped and used in the procedure.

Exhibit 22-29 indicates the appropriate mapping of O-D demand volumes into phase movement volumes for the eight covered interchange types. The designation of the O-D demands is shown in Exhibit 22-21. The mapped phase movement volumes are then used in Step 2 to compute critical flow ratios.

Step 2: Computation of Critical Flow Ratios

The subsections that follow detail the computation of the critical flow ratio Y_c for the interchange for the eight basic configurations covered by this methodology.

Exhibit 22-29
Mapping of Interchange Origins and Destinations into Phase Movements for Operational Interchange Type Selection Analysis

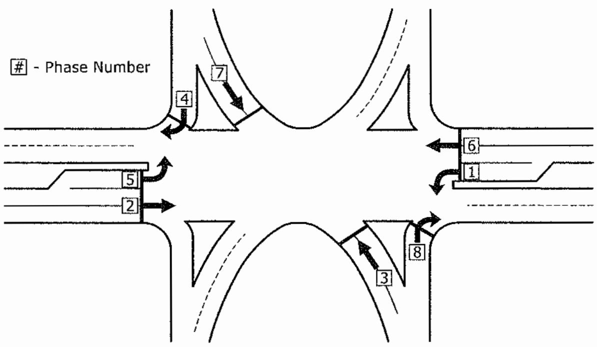
Interchange Type	NEMA Phase Movement Number 1	2	3	4	5	6	7	8
SPUI	H	I+**F**	A+M	**C**	E	J+**G**	D+N	**B**
TUDI /CUDI	H+M	E+I+**F**	--	D+**C**+N	E+N	H+J+**G**	--	A+M+**B**
CDI (I)	H+M	E+I+**F**	--	D+**C**+N	--	J+A	--	--
CDI (II)	--	I+D	--	--	E+N	H+J+**G**	--	A+M+**B**
Parclo A-4Q (I)	--	E+I	--	D+N+**C**	--	J+A+**M**+**H**	--	--
Parclo A-4Q (II)	--	I+D+**N**+**E**	--	--	--	J+H	--	A+M+**B**
Parclo A-2Q (I)	--	E+I	--	D+N+**C**	F	J+A+**H**+**M**	--	--
Parclo A-2Q (II)	G	I+D+**E**+**N**	--	--	--	H+J	--	A+M+**B**
Parclo B-4Q (I)	H+M	I+E+**F**	--	--	--	J+A	--	--
Parclo B-4Q (II)	--	I+D	--	--	E+N	H+J+**G**	--	--
Parclo B-2Q (I)	H+M	E+I+**F**	--	--	--	J+A	--	C
Parclo B-2Q (II)	--	I+D	--	B	E+N	H+J+**G**	--	--

Notes: -- indicates that phase movement does not exist for this interchange configuration.
Bold indicates movements not included when they operate from a separate lane with YIELD or STOP control.

Single-Point Urban Interchange

The phase movements in a SPUI are illustrated in Exhibit 22-30.

Exhibit 22-30
Phase Movements in a SPUI



Source: Bonneson et al. (*4*).

The sum of critical flow ratios is estimated as follows:

Equation 22-26

$$Y_c = A + R$$

$$A = \max\left[\left(\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2}\right); \left(\frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6}\right)\right]$$

$$R = \max\left[\left(\frac{v_3}{s_3 n_3} + \frac{v_4}{s_4 n_4}\right); \left(\frac{v_7}{s_7 n_7} + \frac{v_8}{s_8 n_8}\right)\right]$$

where

Y_c = sum of the critical flow ratios,

v_i = phase movement volume for phase i (veh/h),

n_i = number of lanes serving phase movement i,

s_i = saturation flow rate for phase movement i (veh/hg/ln),

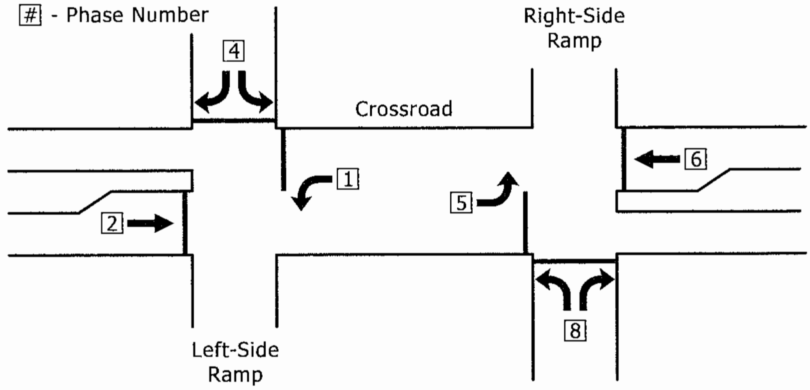
A = critical flow ratio for the arterial movements, and

R = critical flow ratio for the exit-ramp movements.

Tight Urban Diamond Interchange

Phase movements in a TUDI are illustrated in Exhibit 22-31.

Exhibit 22-31
Phase Movements in a Tight Urban or Compressed Urban Diamond Interchange



Source: Bonneson et al. (*4*).

The sum of critical flow ratios is computed as follows:

Equation 22-27

$$Y_c = A + R$$

$$A = \max\left[\left(\frac{v_2}{s_2 n_2} + \frac{v_4}{s_4 n_4} - y_3\right); \left(\frac{v_5}{s_5 n_5} + y_7\right)\right]$$

$$R = \max\left[\left(\frac{v_1}{s_1 n_1} + y_3\right); \left(\frac{v_6}{s_6 n_6} + \frac{v_8}{s_8 n_8} - y_7\right)\right]$$

$$y_3 = \min\left[\frac{v_4}{s_4 n_4}; y_t\right]$$

$$y_7 = \min\left[\frac{v_8}{s_8 n_8}; y_t\right]$$

where y_3 and y_7 are the effective flow ratios for concurrent (or transition) Phases 3 and 7, respectively; y_t is the effective flow ratio for the concurrent phase when dictated by travel time; and other variables are as previously defined.

For preliminary design applications, the default values of Exhibit 22-32 are recommended for y_t. The distance between the two intersections is measured from the centerline of the left ramp roadway to the centerline of the right ramp roadway.

Exhibit 22-32
Default Values for y_t

Distance Between Intersections D'	Default Value for y_t
200 ft	0.050
300 ft	0.070
400 ft	0.085

For Phase Movements 2 and 6, the number of assigned lanes (n_2 and n_6) is related to the arterial left-turn bay design. If the left-turn bay extends back to the external approach to the interchange, then the number of lanes on these external approaches is the total number of approaching lanes, including the left-turn bay. If the left-turn bay is provided only on the internal arterial link, n_2 or n_6, or both, would not include this lane.

Compressed Urban Diamond Interchange

Exhibit 22-31 illustrates the phase movement volumes for a CUDI. They are the same as for a TUDI. The sum of critical flow ratios is computed as follows:

$$Y_c = A + R$$

$$A = \max\left[\left(\frac{v_1}{s_1 n_1} + y_2\right);\left(\frac{v_5}{s_5 n_5} + y_6\right)\right]$$

$$R = \max\left[\left(\frac{v_4}{s_4 n_4}\right);\left(\frac{v_8}{s_8 n_8}\right)\right]$$

$$y_2 = \max\left[\left(\frac{v_2}{s_2 n_2}\right);\left(\frac{v_5}{s_2}\right)\right]$$

$$y_6 = \max\left[\left(\frac{v_8}{s_8 n_8}\right);\left(\frac{v_1}{s_6}\right)\right]$$

Equation 22-28

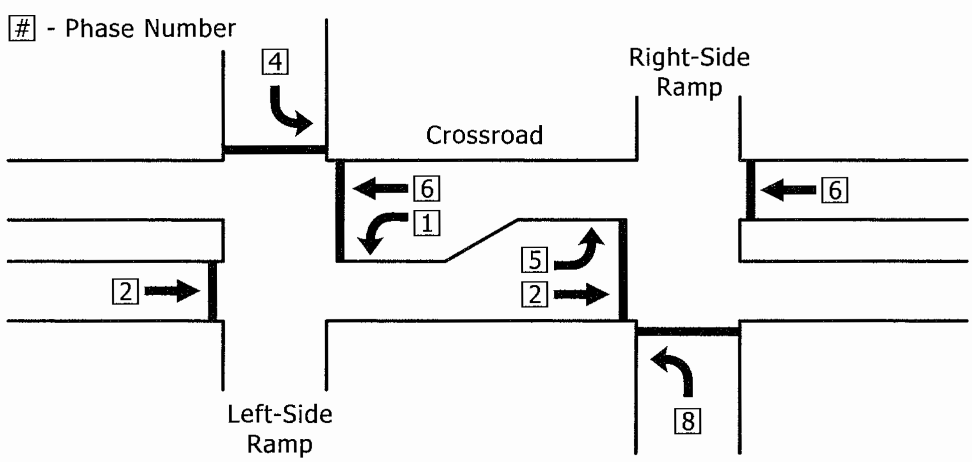
where y_2 and y_6 are the flow ratios for Phases 2 and 6, respectively, with consideration of prepositioning, and all other variables are as previously defined.

All Interchanges with Two Signalized Intersections and Separate Controllers

These interchange types include CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q. The computation of the maximum sum of critical volumes is the same for each. Each has two signalized intersections, and each is generally operated with two controllers.

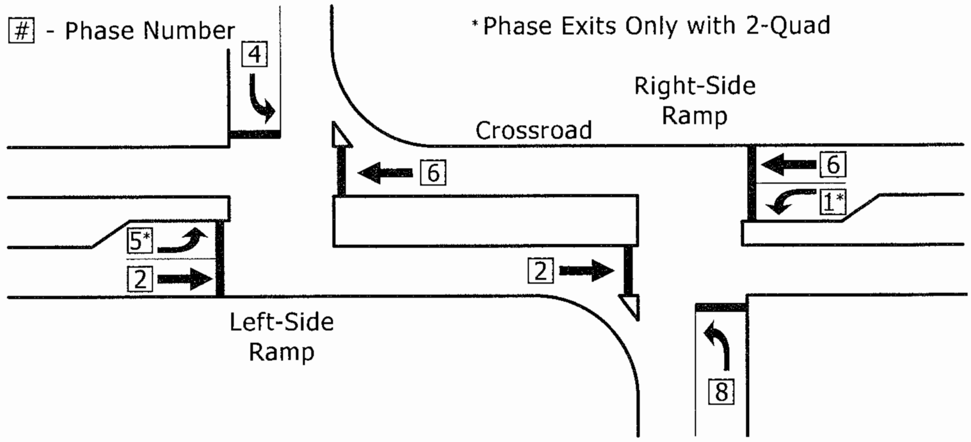
While the equations for estimating the maximum sum of critical volumes are the same, the phase movement volumes differ for each type of interchange, as was indicated in Exhibit 22-29. Exhibit 22-33 through Exhibit 22-35 illustrate the phase movements for each of these interchange types.

Exhibit 22-33
Phase Movements in a CDI



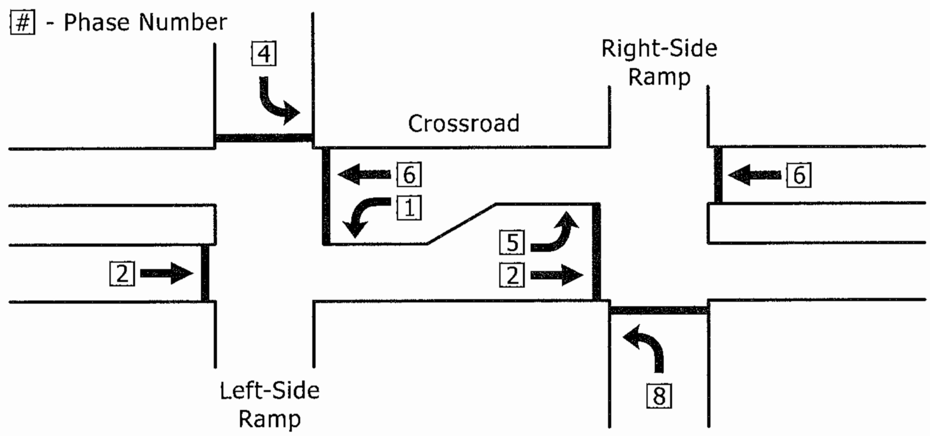
Source: Bonneson et al. (*4*).

Exhibit 22-34
Phase Movements in Parclo A-2Q and A-4Q Interchanges



Source: Messer and Bonneson (*3*).

Exhibit 22-35
Phase Movements in Parclo B-2Q and B-4Q Interchanges



Source: Messer and Bonneson (*3*).

For all conventional diamond, Parclo A, and Parclo B interchanges, the sum of critical flow ratios is computed as follows:

Equation 22-29

$$Y_{c,\max} = \max\left[Y_{c,I}; Y_{c,II}\right]$$

$$Y_{c,I} = A_I + R_I$$

$$Y_{c,II} = A_{II} + R_{II}$$

$$A_{I,II} = \max\left[\left(\frac{v_1}{s_1 n_1} + \frac{v_2}{s_2 n_2}\right); \left(\frac{v_5}{s_5 n_5} + \frac{v_6}{s_6 n_6}\right)\right]$$

$$R_{I,II} = \max\left[\left(\frac{v_4}{s_4 n_4}\right); \left(\frac{v_8}{s_8 n_8}\right)\right]$$

where all variables are as previously defined.

Note that when values of A_I, A_{II}, R_I, and R_{II} are computed, the movement volumes vary for the Intersections I and II, even though the phase movement designations are the same (Exhibit 22-29).

Some of the phase movement volumes do not exist in either Intersection I or II. A value of 0 is used for the volume in each case where this occurs.

Step 3: Estimation of Interchange Delay

Interchange delay for each interchange type or design is estimated by using regression models that were developed primarily from simulation output but validated with a limited amount of field data (*4*). In each case, two delay estimators are provided on the basis of the control of the off-ramp right-turn movements:

- Case A: Used where the right-turn movements from freeway off-ramps are controlled by the signal.
- Case B: Used where the right-turn movements from freeway off-ramps have a separate lane or lanes that are either free (uncontrolled) or controlled by a YIELD sign.

For SPUIs, a third condition is added. Where the right turns from the freeway ramps are controlled by a signal and right-turn-on-red is allowed, both cases are used, and the results are weighted by the proportions of right turns made during the red and green indications. Since the signal timing is unknown for an interchange type selection application, it is recommended that a 50%/50% split be assumed.

This modification, applied only to SPUIs, is necessary due to difficulties experienced in simulating right-turn-on-red at these interchanges.

Exhibit 22-36 gives the delay equations used to estimate interchange delay for the eight interchange types covered by the interchange type selection procedure. In each case, the variables used are defined as follows:

d = interchange delay (s/veh);

Y_c = critical or controlling flow ratio from Step 1; and

D = distance between the two intersections, measured between the centerlines of the two ramp roadways along the surface arterial (ft).

Exhibit 22-36 also shows the ranges of D' over which these equations are valid. They generally represent the normal design range for these interchange types. These equations should be used with great caution beyond these ranges.

Exhibit 22-36
Estimation of Interchange Delay for Eight Basic Interchange Types

Interchange Type	Valid Range of *D'* (ft)	Interchange Delay (d_I) CASE A: Right Turns Signalized	CASE B: Right Turns Free or YIELD-Controlled
SPUI	150–400	$15.1+(0.01D+16.0)\left(\frac{Y_c}{1-Y_c}\right)$	$15.1+(0.008\,D+5.9)\left(\frac{Y_c}{1-Y_c}\right)$
TUDI	200–400	$13.4+14.2\left(\frac{Y_c}{1-Y_c}\right)$	$13.4+12.8\left(\frac{Y_c}{1-Y_c}\right)$
CUDI	600–800	$19.2+[9.4-0.011\,(D-700)]\left[\frac{Y_c}{1-Y_c}\right]$	$19.2+[8.6-0.009(D-700)]\left[\frac{Y_c}{1-Y_c}\right]$
CDI	900–1,300	$17.1+[5.0-0.011(D-1{,}100)]\left[\frac{Y_c}{1-Y_c}\right]$	$17.1+[4.6-0.009\,(D-1{,}100)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo A-4Q	700–1,000	$11.7+[7.8-0.011(D-800)]\left[\frac{Y_c}{1-Y_c}\right]$	$11.7+[6.6-0.009\,(D-800)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo A-2Q	700–1,000	$19.1+[8.3-0.011(D-800)]\left[\frac{Y}{1-Y_c}\right]$	$19.1+[8.3-0.009\,(D-800)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo B-4Q	1,000–1,400	$9.3+[3.5-0.011(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$	$9.3+[3.4-0.009\,(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$
Parclo B-2Q	1,000–1,400	$26.2+[3.9-0.011(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$	$26.2+[3.2-0.009\,(D-1{,}200)]\left[\frac{Y_c}{1-Y_c}\right]$

Delay estimates can be related to LOS. For consistency, the same criteria as used for the operational analysis methodology (Exhibit 22-11) are applied. Because LOS F is based on a *v/c* ratio greater than 1.00 or a queue storage ratio greater than 1.00, this interchange type selection methodology will never predict LOS F, because it does not predict these ratios. Users should be exceedingly cautious of results when interchange delay exceeds 85 to 90 s/veh.

In evaluating alternative interchange types, the exact distance, *D'*, may not be known for each of the alternatives. It is recommended that all lengths be selected at the midpoint of the range shown in Exhibit 22-36 for this level of analysis.

Interpretation of Results

The output of the interchange type selection procedure for signalized interchanges is a set of delay predictions for (*a*) various interchange types, (*b*) various distances *D* between the two intersections, or (*c*) various numbers and assignments of lanes on ramps and the surface arterials.

While, in general, the lower the interchange delay the better, a final choice must consider a number of other criteria that are not part of this methodology, including the following:

- Availability of right-of-way,
- Environmental impacts,
- Social impacts,
- Construction cost, and
- Benefit–cost analysis.

This methodology provides valuable information that can be used, in conjunction with other analyses, in making an appropriate choice of an interchange type and some of the primary design parameters. The final design, however, will be based on many other criteria in addition to the output of this methodology.

Users are also cautioned that while the definition of interchange delay is similar for both the interchange type selection methodology and the operational analysis methodology, different modeling approaches to delay prediction were taken, and there is no guarantee that the results of the two methodologies will be consistent.

3. APPLICATIONS

DEFAULT VALUES

Agencies that use the methodologies of this chapter are encouraged to develop a set of local default values based on measurements at interchanges in their jurisdiction. Local default values provide the best means of ensuring reasonable accuracy in the analysis results. In the absence of local default values, the values provided in Chapter 18, Signalized Intersections, can be used if appropriate.

TYPES OF ANALYSIS

There are two general types of analysis for signalized interchange ramp terminals: (*a*) final design and traffic operational analysis and (*b*) operational analysis for interchange type selection. Planning applications, as defined in other methodological chapters in the HCM, are not included in this chapter, because signal timing is a critical input to the procedure and is typically not known in the planning stages of a project.

In a final design and operational analysis the interchange configuration is given and detailed traffic control information is available. During the planning and preliminary design analysis stage, however, the engineer typically considers several configurations (e.g., SPUI, diamond), with limited input information available (for example, minimal or no traffic control information). At that stage, the interchange type selection analysis should be used to determine which interchange configurations would be preferable from an operational perspective. These two general types of analysis have different objectives, different requirements, and different levels of required input, and each of them is defined and discussed below.

Final Design and Operational Analysis

Final design and operational analysis for signalized interchanges is to be conducted when the type of interchange is known. The objective is either to provide final design details for LOS or to assess the interchange and provide LOS and other performance measures. Two subcategories are distinguished: (*a*) design analysis (where the input is the desired LOS and the outputs are design elements) and (*b*) operational analysis (where the input is complete design and the output is LOS).

Design analyses include highway design and signal design and are concerned with the physical, geometric, and signal control characteristics of the facility so that it operates at a desired LOS. For those types of analysis, the evaluation is conducted iteratively. The input data typically required for design analysis are fairly detailed and based substantially on design attributes that are being proposed. The objective of the interchange design analysis is to recommend geometric elements such as the number of lanes and storage bay length, or a signal control scheme, to maintain a given LOS. The principal inputs for design analysis are the design hourly volumes and the desired LOS for a given interchange configuration.

The objective of operational analysis is to obtain the LOS of a facility under given traffic, design, and signal control conditions. As in design analysis, the operational analysis is conducted for a given interchange configuration, where the input data include the turning movement demands, number of lanes and their respective lengths and channelization, and traffic control information.

Operational Analysis for Interchange Type Selection

This type of analysis should be used when the type of interchange is not known yet and the engineer is interested in assessing the traffic operations of various alternatives. Detailed information is not known (e.g., signalization information, design details). The principal inputs for an interchange type selection analysis are O-D demands and a list of feasible configurations that can be tested according to site physical and right-of-way conditions. This type of analysis considers signalized interchanges but does not consider unsignalized interchanges or interchanges with roundabouts.

O-D and Turning Movements

Exhibit 22-37 through Exhibit 22-44 illustrate how O-D movements can be obtained from turning movements for each type of interchange considered in this methodology. Exhibit 22-45 through Exhibit 22-52 provide the corresponding calculations for obtaining turning movements from O-D movements.

Exhibit 22-37
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo A-2Q Interchanges

Input					Output	
	Intersection I		Intersection II			
Approach	Turning Move-ment	Volume (veh/h)	Turning Move-ment	Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	EXT-LT		LT		A = (NB LT) – (NB UT)	
	RT		INT-RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	LT		EXT-LT		D = (SB LT) – (SB UT)	
	INT-RT		RT		E = (EB INT-RT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-LT	
Northbound (NB)	LT		LT		G = WB EXT-LT	
	RT		RT		H = (WB INT-RT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (SB UT and NB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-38
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo A-4Q Interchanges

Approach	Intersection I Turning Movement	Intersection I Volume (veh/h)	Intersection II Turning Movement	Intersection II Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
	Input				**Output**	
Eastbound (EB)	LT		LT		A = (NB LT) – (NB UT)	
	EXT-RT		INT-RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	LT		LT		D = (SB LT) – (SB UT)	
	INT-RT		EXT-RT		E = (EB INT-RT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT		G = WB EXT-RT	
	RT		RT		H = (WB INT-RT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (SB UT and NB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-39
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo AB-2Q Interchanges

Approach	Intersection I Turning Movement	Intersection I Volume (veh/h)	Intersection II Turning Movement	Intersection II Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
	Input				**Output**	
Eastbound (EB)	LT		LT		A = (NB LT(II)) – (NB UT(II))	
	EXT-RT		INT-RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = NB LT(I)	
Westbound (WB)	INT-LT		EXT-LT		D = (NB RT(I)) – (NB UT(I))	
	RT		RT		E = (EB INT-RT) – (NB UT(I))	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT(I)		LT(II)		G = WB EXT-LT	
	RT(I)		RT(II)		H = (WB INT-LT) – (NB UT(II))	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT(I))	
	UT(I)		UT(II)		J = (WB INT-TH)–(NB LT(II))+(NB UT(II))	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = NB UT(II)	
	UT		UT		N = NB UT(I)	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway [NB UT(I) and NB UT(II)] are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-40
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo AB-4Q Interchanges

Approach	Intersection I Turning Movement	Intersection I Volume (veh/h)	Intersection II Turning Movement	Intersection II Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
	Input				**Output**	
Eastbound (EB)	LT		LT		A = (NB LT(II)) – (NB UT(II))	
	EXT-RT		INT-RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = SB RT(I)	
Westbound (WB)	INT-LT		LT		D = (NB RT(I)) – (NB UT(I))	
	RT		EXT-RT		E = (EB INT-RT) – (NB UT(I))	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT(II)		G = WB EXT-LT	
	RT(I)		RT(II)		H = (WB INT-LT) – (NB UT(II))	
	TH		TH		I = (EB INT-TH)–(NB RT(I)) + (NB UT(I))	
	UT(I)		UT(II)		J = (WB INT-TH) – (NB LT(II)) + (NB UT(II))	
Southbound (SB)	LT		LT		K	
	RT(I)		RT		L	
	TH		TH		M = NB UT(II)	
	UT		UT		N = NB UT(I)	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway [NB UT(I) and NB UT(II)] are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-41
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo B-2Q Interchanges

Approach	Intersection I Turning Movement	Intersection I Volume (veh/h)	Intersection II Turning Movement	Intersection II Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (SB RT) – (SB UT)	
	EXT-RT		RT		B = SB LT	
	EXT-TH		INT-TH		C = NB LT	
Westbound (WB)	INT-LT		LT		D = (NB RT) – (NB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (NB UT)	
	INT-TH		EXT-TH		F = (EB EXT-RT)	
Northbound (NB)	LT		LT		G = (WB EXT-RT)	
	RT		RT		H = (WB INT-LT) – (SB UT)	
	TH		TH		I = (EB INT-TH) – (NB RT) + (NB UT)	
	UT		UT		J = (WB INT-TH) – (SB RT) + (SB UT)	
Southbound (SB)	LT		LT		K	
	RT		RT		L	
	TH		TH		M = SB UT	
	UT		UT		N = NB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-42
Worksheet for Obtaining O-D Movements from Turning Movements for Parclo B-4Q Interchanges

Approach	Intersection I Turning Movement	Intersection I Volume (veh/h)	Intersection II Turning Movement	Intersection II Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (SB RT(II)) – (SB UT)	
	EXT-RT		RT		B = NB RT(II)	
	EXT-TH		INT-TH		C = SB RT(I)	
Westbound (WB)	INT-LT		LT		D = (NB RT(I)) – (NB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (NB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT		G = WB EXT-RT	
	RT(I)		RT(II)		H = (WB INT-LT) – (SB UT)	
	TH		TH		I = (EB INT-TH) – (NB RT(I)) + (NB UT)	
	UT		UT		J = (WB INT-TH) – (SB RT(II)) + (SB UT)	
Southbound (SB)	LT		LT		K	
	RT(I)		RT(II)		L	
	TH		TH		M = SB UT	
	UT		UT		N = NB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-43
Worksheet for Obtaining O-D Movements from Turning Movements for Diamond Interchanges

Approach	Intersection I Turning Movement	Intersection I Volume (veh/h)	Intersection II Turning Movement	Intersection II Volume (veh/h)	O-D Movement Calculation	Volume (veh/h)
Eastbound (EB)	LT		INT-LT		A = (NB LT) – (NB UT)	
	EXT-RT		RT		B = NB RT	
	EXT-TH		INT-TH		C = SB RT	
Westbound (WB)	INT-LT		LT		D = (SB LT) – (SB UT)	
	RT		EXT-RT		E = (EB INT-LT) – (SB UT)	
	INT-TH		EXT-TH		F = EB EXT-RT	
Northbound (NB)	LT		LT		G = WB EXT-RT	
	RT		RT		H = (WB INT-LT) – (NB UT)	
	TH		TH		I = (EB INT-TH) – (SB LT) + (SB UT)	
	UT		UT		J = (WB INT-TH) – (NB LT) + (NB UT)	
Southbound (SB)	LT		LT		K = NB TH	
	RT		RT		L = SB TH	
	TH		TH		M = NB UT	
	UT		UT		N = SB UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
The flows of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-44
Worksheet for Obtaining O-D Movements from Turning Movements for SPUIs

Input			Output	
Approach	**Turning Movement**	**Volume (veh/h)**	**O-D Movement Calculation**	**Volume (veh/h)**
Eastbound (EB)	LT		A = NB LT	
	RT		B = NB RT	
	TH		C = SB RT	
Westbound (WB)	LT		D = SB LT	
	RT		E = EB LT	
	TH		F = EB RT	
Northbound (NB)	LT		G = WB RT	
	RT		H = WB LT	
	TH		I = EB TH	
	UT		J = WB TH	
Southbound (SB)	LT		K = NB TH	
	RT		L = SB TH	
	TH		M	
	UT		N	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through.
The flow of the two U-turn movements from the freeway (NB UT and SB UT) are user-specified.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-45
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo A-2Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	EXT-LT = F		LT	
B			RT		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	LT		EXT-LT = G	
E			INT-RT = H+M		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-46
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo A-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Move-ment	**Volume (veh/h)**	**Approach**	**Turning Movement Calculation**	**Volume (veh/h)**	**Turning Movement Calculation**	**Volume (veh/h)**
A		Eastbound (EB)	EXT-LT = F		LT	
B			RT		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	LT		EXT-LT = G	
E			INT-RT = H+M		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-47
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-2Q Interchanges

Input		Output				
O-D Movement	Volume (veh/h)	Approach	Intersection I Turning Movement Calculation	Volume (veh/h)	Intersection II Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		EXT-LT = G	
E			RT		RT	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT(I) = C		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-48
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo AB-4Q Interchanges

Input		Output				
O-D Movement	Volume (veh/h)	Approach	Intersection I Turning Movement Calculation	Volume (veh/h)	Intersection II Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		LT	
B			EXT RT = F		INT-RT = E+N	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT(II) = A+M	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT(I) = N		UT(II) = M	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT	
M			TH		TH	
N			UT		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-49
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-2Q Interchanges

Input		Output				
O-D Movement	Volume (veh/h)	Approach	Intersection I Turning Movement Calculation	Volume (veh/h)	Intersection II Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT = C		LT	
H			RT = D+N		RT	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT = B	
L			RT		RT = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-50
Worksheet for Obtaining Turning Movements from O-D Movements for Parclo B-4Q Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT	
H			RT(I) = D+N		RT(II) = B	
I			TH		TH	
J			UT = N		UT	
K		Southbound (SB)	LT		LT	
L			RT(I) = C		RT(II) = A+M	
M			TH		TH	
N			UT		UT = M	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-51
Worksheet for Obtaining Turning Movements from O-D Movements for Diamond Interchanges

Input		Output				
			Intersection I		Intersection II	
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT		INT-LT = E+N	
B			EXT RT = F		RT	
C			EXT-TH = I+E		INT-TH = I+D	
D		Westbound (WB)	INT-LT = H+M		LT	
E			RT		EXT-RT = G	
F			INT-TH = J+A		EXT-TH = J+H	
G		Northbound (NB)	LT		LT = A+M	
H			RT		RT = B	
I			TH		TH = K	
J			UT		UT = M	
K		Southbound (SB)	LT = D+N		LT	
L			RT = C		RT	
M			TH = L		TH	
N			UT = N		UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

Exhibit 22-52
Worksheet for Obtaining Turning Movements from O-D Movements for SPUIs

Input		Output		
O-D Movement	Volume (veh/h)	Approach	Turning Movement Calculation	Volume (veh/h)
A		Eastbound (EB)	LT = E	
B			RT = F	
C			TH = I	
D		Westbound (WB)	LT = H	
E			RT = G	
F			TH = J	
G		Northbound (NB)	LT = A	
H			RT = B	
I			TH = K	
J			UT	
K		Southbound (SB)	LT = D	
L			RT = C	
M			TH = L	
N			UT	

Notes: LT = left turn, RT = right turn, UT = U-turn, TH = through, INT = internal, EXT = external.
Shading indicates movements that do not occur in this interchange form.

USE OF ALTERNATIVE TOOLS

General guidance on alternative tools is provided in Chapters 6 and 7.

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This section contains specific guidance for the application of alternative tools to the analysis of interchange ramp terminals. Chapter 34, Interchange Ramp Terminals: Supplemental, contains supplemental examples illustrating the use of alternative tools for interchange analysis. Additional information on this topic may be found in the Technical Reference Library in Volume 4.

As indicated in Chapter 6, traffic models may be classified in several ways (e.g., deterministic versus stochastic, macroscopic versus microscopic). The alternative tools used for interchange analysis are generally based on models that are microscopic and stochastic in nature. Therefore, the discussion in this section will be limited to microsimulation tools.

Strengths of the HCM Procedure

This chapter offers a comprehensive procedure for analyzing the performance of several types of interchanges. Simulation-based tools offer a more detailed treatment of the arrival and departure of individual vehicles and features of the signal control system, but for most purposes, the HCM procedure produces an acceptable approximation. The HCM procedure offers some advantages over the simulation approach:

- The HCM provides saturation flow rate adjustment factors based on extensive field studies.
- The HCM produces direct estimates of capacity and *v/c* ratio. These measures are much more elusive in simulation.
- The HCM provides LOS by O-D, which facilitates the comparison of operational performance for different interchange configurations.
- It provides deterministic estimates of the measures of effectiveness, which is important for some purposes such as development impact review.
- Simulation tools use definitions of delay (and therefore LOS) different from those of the HCM, especially for movements that are oversaturated at some point during the analysis. Great care must therefore be taken in producing LOS estimates directly from simulation. Chapter 7 discusses simulation-based performance measures in more detail.

Identified Limitations of the HCM Procedure

The identified limitations of the HCM procedure for this chapter cover a number of conditions that are not evaluated explicitly, including the following:

- Oversaturated conditions, particularly cases when the downstream queue spills back into the upstream intersection for long periods of time;
- The impact of spillback on freeway operations (however, the method does estimate the expected queue storage ratio for the ramp approaches);
- Ramp metering and its resulting spillback of vehicles into the interchange;

- Impacts of the interchange operations on arterial operations and the extended surface street network;
- Lane utilizations for interchanges with additional approaches that are not part of the prescribed interchange configuration; and
- Full cloverleaf interchanges (freeway-to-freeway or system interchanges), since the scope of the chapter is limited to service interchanges (e.g., freeway-to-arterial interchanges).

If any of these conditions apply to a particular situation, then alternative tools that recognize them explicitly should be considered to supplement or replace the methodology described in this chapter.

Additional Features and Performance Measures Available From Alternative Tools

This chapter provides a methodology for estimating the capacity, control delay, queue storage ratio, and LOS associated with a given set of traffic, control, and design conditions at an interchange. As with most other procedural chapters in this manual, simulation outputs, especially graphics-based presentations, can provide details on problems at specific elements of the interchange that might otherwise go unnoticed with a macroscopic analysis. For example, problems associated with turn bay overflow or blockage of access to turn bays can be better observed by using microscopic simulation tools. Alternative tools offer additional performance measures such as number of stops, fuel consumption, and pollution. The animated graphics displays offered by many simulation tools are especially useful for observing network operations and identifying problems at specific elements.

Development of HCM-Compatible Performance Measures Using Alternative Tools

Simulation tools provide a wealth of information with regard to performance measures, including queue length, travel time, emissions, and so forth. However, simulation tools often have different definitions for each of these performance measures. General guidance on developing compatible performance measures based on the analysis of individual vehicle trajectories is provided in Chapter 7, with supplemental examples provided in Chapter 24, Concepts: Supplemental. Chapter 18, Signalized Intersections, provides some specific guidance on performance measures for signalized approaches that also applies to this chapter. To obtain LOS for a specific O-D, the analyst will need to obtain the performance measures for the specific approaches using that particular O-D and aggregate them as indicated in the methodology section of this chapter.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

For interchanges, the definitions of delay and queuing are the most significant conceptual differences between the HCM and simulation modeling. Both are measures of effectiveness used to obtain LOS for each O-D, and simulated estimates of them would produce results inherently different from those obtained by the analytical method described in this chapter.

Lane utilization is also treated differently. Simulation tools derive lane distributions and utilization implicitly from driver behavior modeling, while the deterministic model used in this chapter develops lane distributions from empirical models. Differences in the treatment of random arrivals are also an issue in the comparison of performance measures. This topic was discussed in detail in Chapter 18, and the same phenomena apply to this chapter.

In some cases, and when saturation flow rate is not an input, simulation tools do not explicitly account for differences between left-turning, through, and right-turning movements, and all three have very similar saturation headway values. Thus, the left- and right-turn lane capacity would likely be overestimated in those types of tools.

Adjustment of Simulation Parameters to Match the HCM Parameters

Some adjustments will generally be required before an alternative tool can be used effectively to supplement or replace the procedures described in this chapter. For example, the parameters that determine the capacity of a signalized approach (e.g., steady state headway and start-up lost time) should be adjusted to ensure that the simulated approach capacities match the HCM values.

One parameter specific to this chapter is the lane utilization on the approaches within the interchange. Driver behavior model parameters that affect lane choice should be examined closely and modified if necessary to produce better agreement with the lane distributions estimated by the procedures in this chapter.

Simulation tools do not produce explicit capacity estimates. The accepted method of determining the capacity of a signalized approach by simulation is to perform the simulation run(s) with a demand in excess of the computed capacity and use the throughput as an indication of capacity. Chapter 7 provides additional guidance on the determination of capacity in this manner. The Chapter 7 discussion points out the complexities that can arise when self-aggravating phenomena occur as the operation approaches capacity. Because of the interaction of traffic movements within an interchange, the potential for self-aggravating situations is especially high.

In complex situations, conceptual differences between the deterministic procedures in this chapter and those of simulation tools may exist such that the production of compatible capacity estimates is not possible. In such cases, the capacity differences should be noted.

Step-by-Step Recommendations for Applying Alternative Tools

General guidance on selecting and applying alternative tools is provided in Chapters 6 and 7. Chapter 18 provides recommendations specifically for signalized intersections that also apply to interchange ramp terminals.

One step that is specific to this chapter is the emulation of the traffic control hardware. Generally, simulation tools provide great flexibility in emulating actuated control, particularly in the type and location of detectors. In most cases, simulation tools attach a controller to each intersection (or node) in the network. This creates problems for some interchange operations in which a controller at one node must be connected to an approach to another node. A diamond interchange operating with one controller is an example of the complexities that can arise in the emulation of the traffic control scheme.

Some tools are able to accommodate complex schemes more flexibly than others. The ability to emulate the desired traffic control scheme is an important consideration in the selection of a tool for interchange analysis.

Sample Calculations Illustrating Alternative Tool Applications

Supplemental problems involving the use of alternative tools for signalized intersection analysis are presented in Chapter 34.

Example Problem 1 in this chapter involves a diamond interchange that offers the potential for illustrating the use of alternative tools. There are no limitations in this example that suggest the need for alternative tools. It is possible, however, to introduce situations in which alternative tools might be needed for a proper assessment of performance.

Chapter 34 includes supplemental examples that apply alternative tools to deal with two conditions that are beyond the scope of the procedures presented in this chapter.

1. A two-way STOP-controlled intersection in close proximity to the diamond interchange and
2. Ramp metering on one of the freeway entrance ramps connected to the interchange.

In both cases, the demand volumes are varied to examine the self-aggravating effects on the operation of the facility.

4. EXAMPLE PROBLEMS

INTRODUCTION

This part of the chapter describes the application of each of the final design, operational analysis for interchange type selection, and roundabouts analysis methods through the use of example problems. Exhibit 22-53 describes each of the three example problems included in this chapter and indicates the methodology applied. Additional example problems are provided in Chapter 34, Interchange Ramp Terminals: Supplemental.

Exhibit 22-53
Example Problem Descriptions

Problem Number	Description	Application
1	Find the control delay, queue storage ratio, and LOS of a diamond interchange	Operational analysis
2	Find the control delay, queue storage ratio, and LOS of a Parclo A-2Q interchange	Operational analysis
3	Compare eight types of signalized interchanges	Operational analysis for interchange type selection

EXAMPLE PROBLEM 1: DIAMOND INTERCHANGE

The Interchange

The interchange of I-99 (northbound/southbound, NB/SB) and University Drive (eastbound/westbound, EB/WB) is a diamond interchange. Exhibit 22-54 provides the interchange volumes and channelization, while Exhibit 22-55 provides the signalization information.

Exhibit 22-54
Example Problem 1: Interchange Volumes and Channelization

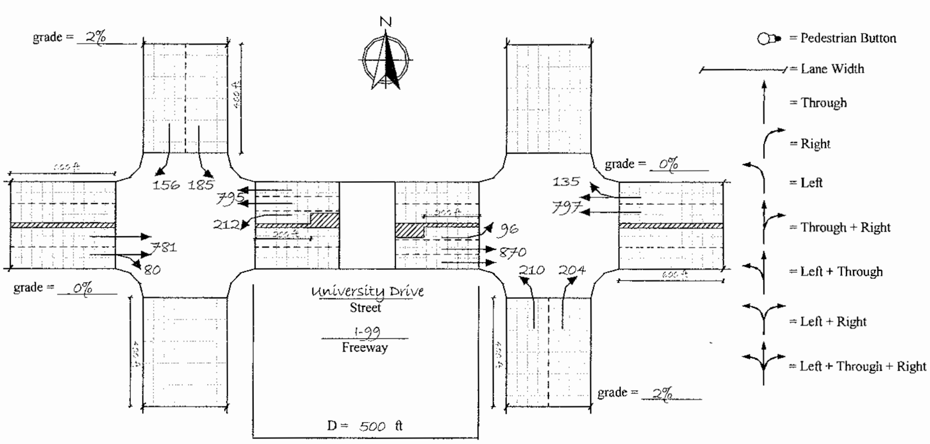


Exhibit 22-55
Example Problem 1: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (4+8)	Φ (4+7)	Φ (2+5)	Φ (4+8)	Φ (1+6)	Φ (3+8)
Green time (s)	63	43	39	63	53	29
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		19			9	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. Travel path radii are 50 ft for all right-turning movements and 75 ft for all left-turning movements. Arrival Type 4 is assumed for all arterial movements and Arrival Type 3 for all other movements.

There are 5% heavy vehicles on both the external and internal through movements, and the peak hour factor (PHF) for the interchange is estimated to be 0.90. Start-up lost time and extension of effective green are both 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Outline of Solution

Calculation of O-Ds

O-Ds through this diamond interchange are calculated on the basis of Exhibit 22-21(a). Since all movements utilize the signal and no right-turns-on-red are allowed, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 22-56.

Exhibit 22-56
Example Problem 1: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	210	233
B	204	227
C	156	173
D	185	206
E	96	107
F	80	89
G	135	150
H	212	236
I	685	761
J	585	650
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

Both external approaches to this interchange consist of a two-lane shared right and through lane group. Use of the two-lane model of Exhibit 22-16 results in the predicted lane utilization percentages for the external through approaches that are presented in Exhibit 22-57.

Approach	V_1	V_2	Maximum Lane Utilization	Lane Utilization Factor
Eastbound external	0.5056	0.4944	0.5056	0.9890
Westbound external	0.5181	0.4819	0.5181	0.9651

Exhibit 22-57
Example Problem 1: Lane Utilization Adjustment Calculations

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the approaches external to the interchange is obtained as shown above in Exhibit 22-57. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated as indicated in Chapter 18, Signalized Intersections. The estimated saturation flow rates for the eastbound approaches are shown in Exhibit 22-58.

Calculation of Common Green and Lost Time due to Downstream Queue and Demand Starvation

The duration of common green between various movements is calculated next. Exhibit 22-59 provides the beginning and end of each phase for the two intersections, as well as the calculations of common green between various movements at the two intersections.

Exhibit 22-58
Example Problem 1: Calculation of Saturation Flow Rate for Eastbound Approaches

	Eastbound Turning Movements		
	EXT-TH, EXT-RT	INT-TH	INT-LT
Base saturation flow (s_0, in pc/hg/ln)	1,900	1,900	1,900
Number of lanes (N)	2	2	1
Lane width adjustment (f_w)	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.952	0.952	1.000
Grade adjustment (f_g)	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.989	0.952	1.000
Left-turn adjustment (f_{LT})	1.000	1.000	0.930
Right-turn adjustment (f_{RT})	0.999	1.000	1.000
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	0.991	1.000	0.930
Traffic pressure adjustment for lane group (f_v)	1.034	1.036	0.963
Adjusted saturation flow (s, in veh/hg/ln)	3,700	3,568	1,703

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-59
Example Problem 1:
Common Green Calculations

Phase	Intersection I Phase Begin	Intersection I Phase End	Intersection II Phase Begin	Intersection II Phase End	
Phase 1	0	63	150	53	
Phase 2	68	111	58	111	
Phase 3	116	155	116	145	
Movement	**1st Green Time of Cycle Begin**	**1st Green Time of Cycle End**	**2nd Green Time of Cycle Begin**	**2nd Green Time of Cycle End**	**Overlap**
EB EXT-TH	0	63			53
EB INT-TH	150	53	116	150	
WB EXT-TH	150	53			53
WB INT-TH	0	111			
SB RAMP	116	155			34
EB INT-TH	150	53	116	150	
NB RAMP	58	111			53
WB INT-TH	0	111			
WB INT-LT	68	111			0
EB INT-TH	150	53			
EB INT-LT	116	145			0
WB INT-TH	0	111			

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, INT = internal, EXT = external.

The next step involves the calculation of lost time due to downstream queues. First, the queue at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. These numbers are then subtracted from the internal link storage length to determine the storage available at the beginning of the respective upstream phase. Exhibit 22-60 presents the calculation of the downstream queues followed by the calculation of the respective lost time due to those downstream queues.

Exhibit 22-60
Example Problem 1:
Calculation of Lost Time due to Downstream Queues

	EB EXT-TH	SB-LT	WB EXT-TH	NB-LT
V_R or V_A (veh/h)	206	868	233	886
N_R or N_A	1	2	1	2
G_R or G_A (s)	39	63	53	63
G_D (s)	97	97	111	111
C (s)	160	160	160	160
CG_{UD} or CG_{RD} (s)	53	34	53	53
Queue length (Q_A or Q_R) (ft)	0.0	4.1	0.0	0.0
G_R or G_A (s)	63	39	63	53
C (s)	160	160	160	160
D_{QA} or D_{QR} (ft)	500	496	500	500
CG_{UD} or CG_{RD} (s)	53	34	53	53
Additional lost time ($L_{D\text{-}A}$ or $L_{D\text{-}R}$) (s)	0.0	0.0	0.0	0.0
Total lost time t'_L (s)	5.0	5.0	5.0	5.0
Effective green time g' (s)	63.0	39.0	63.0	53.0

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, EXT = external.

Calculation of lost time due to demand starvation begins by determining the queue storage length at the beginning of an interval with demand starvation potential by using Equation 22-17. The lost time due to demand starvation is then calculated by using Equation 22-16. The respective calculations are presented in Exhibit 22-61.

Exhibit 22-61
Example Problem 1: Calculation of Lost Time due to Demand Starvation

	EB INT-TH	WB INT-TH
$v_{Ramp\text{-}L}$ (veh/h)	206	233
$v_{Arterial}$ (veh/h)	868	886
C (s)	160	160
$N_{Ramp\text{-}L}$	1	1
$N_{Arterial}$	2	2
CG_{RD} (s)	34	53
CG_{UD} (s)	53	53
H_I	2.02	2.04
$Q_{INITIAL}$ (ft)	0	0
CG_{DS} (s)	0	0
L_{DS} (s)	0	0
t''_L (s)	5	5
Effective green time g'' (s)	97	111

Note: EB = eastbound, WB = westbound, TH = through, INT = internal.

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 22-62 presents the calculation of the queue storage ratio for all eastbound movements in Example Problem 1. Control delay for each movement is calculated according to Equation 18-47. Exhibit 22-63 provides the control delay for each eastbound movement of the interchange.

Exhibit 22-62
Example Problem 1: Queue Storage Ratio Calculations for Eastbound Movements

	EXT-TH, EXT-RT	INT-LT	INT-TH
Q_{bL} (ft)	0.0	0.0	0.0
v (veh/h/ln group)	957	107	967
s (veh/h/ln)	1850	1703	1784
g (s)	63	29	97
g/C	0.39	0.18	0.61
I	1.00	0.71	0.71
c (veh/h/ln group)	1459	309	2163
$X = v/c$	0.66	0.35	0.45
r_a (ft/s^2)	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0
S_s (mi/h)	5	5	5
S_{pl} (mi/h)	40	40	40
S_a (mi/h)	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04
Rp	1.000	1.333	1.333
P	0.39	0.24	0.81
r (s)	97	131	63
t_f (s)	0.01	0.00	0.00
q (veh/s)	0.27	0.03	0.27
q_g (veh/s)	0.27	0.04	0.36
q_r (veh/s)	0.27	0.03	0.13
Q_1 (veh)	15.22	3.47	3.84
Q_2 (veh)	0.93	0.14	1.37
T	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00
t_A	0	0	0
Q_e (veh)	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00
Q_3 (veh)	0.00	0.00	0.00
Q (veh)	16.15	3.65	3.98
L_h (ft)	25.01	25.00	25.01
L_a (ft)	600	200	500
R_Q	0.67	0.46	0.20

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-63
Example Problem 1: Calculation of Control Delay for Eastbound Movements

	EXT-TH, EXT-RT	INT-LT	INT-TH
g (s)	NA	29	97
g' (s)	63	NA	NA
g/C or g'/C	0.39	0.18	0.61
c (veh/h)	1459	309	2163
$X = v/c$	0.66	0.35	0.45
d_1 (s/veh)	39.6	52.8	7.3
k	0.5	0.5	0.5
d_2 (s/veh)	4.6	2.2	0.5
d_3 (s/veh)	0.0	0.0	0.0
k_{min}	0.04	0.04	0.04
u	0	0	0
t	0	0	0
d (s/veh)	44.1	55.0	7.8

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external, NA = not applicable.

Results

This section discusses the estimation of the delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as a sum of the movement delays for each movement utilized by the O-D, as shown in Equation 22-1. Next, the v/c and queue storage ratios must be checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement will be F. Exhibit 22-64 presents a summary of the results for all O-D movements at this interchange. As shown, neither v/c nor R_Q exceed 1, and all movements have LOS E or better. The LOS is determined by using Exhibit 22-11.

Exhibit 22-64
Example Problem 1: O-D Movement LOS

O-D Movement	Delay (s)	$v/c > 1$?	$R_Q > 1$?	LOS
A	45.6	No	No	C
B	43.7	No	No	C
C	54.6	No	No	C
D	63.6	No	No	D
E	99.2	No	No	E
F	44.2	No	No	C
G	37.5	No	No	C
H	82.7	No	No	D
I	52.0	No	No	C
J	39.8	No	No	C

EXAMPLE PROBLEM 2: PARCLO A-2Q INTERCHANGE

The Interchange

The interchange of I-75 (NB/SB) and Newberry Avenue (EB/WB) is a Parclo A-2Q interchange. Exhibit 22-65 provides the interchange volumes and channelization, while Exhibit 22-66 provides the signalization information.

Exhibit 22-65
Example Problem 2: Intersection Plan View

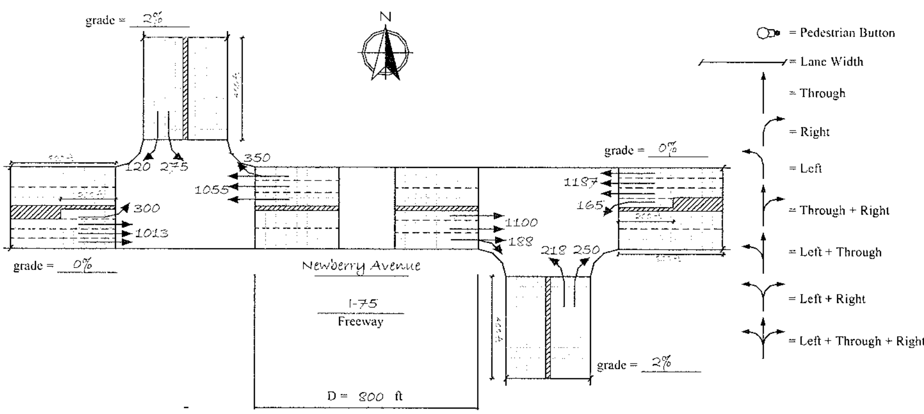


Exhibit 22-66
Example Problem 2: Signalization Information

Phase	Intersection I 1	Intersection I 2	Intersection I 3	Intersection II 1	Intersection II 2	Intersection II 3
NEMA	Φ (3+8)	Φ (4+8)	Φ (2+5)	Φ (4+7)	Φ (1+6)	Φ (4+8)
Green time (s)	25	60	40	25	35	65
Yellow + all red (s)	5	5	5	5	5	5
Offset (s)		0			0	

The Question

What are the control delay, queue storage ratio, and LOS for this interchange?

The Facts

There are no closely spaced intersections to this interchange, and it operates as a pretimed signal with no right-turns-on-red allowed. The eastbound and westbound left-turn radii are 80 ft, while all remaining turning movements have radii of 50 ft. The arrival type is assumed to be 4 for arterial movements and 3 for all other movements.

There are 10% heavy vehicles on both the external and internal through movements, and the PHF for the interchange is estimated to be 0.95. Start-up lost time is 3 s for all approaches, while the extension of effective green is 2 s for all approaches. During the analysis period there is no parking and no buses, bicycles, or pedestrians utilize the interchange.

Outline of Solution

Calculation of O-Ds

O-Ds through this interchange are calculated on the basis of Exhibit 22-21(b). Since all movements utilize the signal and no right-turns-on-red are allowed, O-Ds can be calculated directly from the turning movements at the two intersections. The results of these calculations and the resulting PHF-adjusted values are presented in Exhibit 22-67.

Exhibit 22-67
Example Problem 2: Adjusted O-D Table

O-D Movement	Demand (veh/h)	PHF-Adjusted Demand (veh/h)
A	218	229
B	250	263
C	120	126
D	275	289
E	188	198
F	300	316
G	165	174
H	350	368
I	825	868
J	837	881
K	0	0
L	0	0
M	0	0
N	0	0

Lane Utilization and Saturation Flow Rate Calculations

The external approaches to this interchange consist of a three-lane through lane group for the external approaches. Use of the three-lane model from Exhibit 22-17 results in the predicted lane utilization percentages for the external through approaches presented in Exhibit 22-68.

Exhibit 22-68
Example Problem 2: Lane Utilization Adjustment Calculations

Approach	V_1	V_2	V_3	Maximum Lane Utilization	Lane Utilization Factor
Eastbound	0.2660	0.2791	0.4549	0.4549	0.7328
Westbound	0.2263	0.2472	0.5265	0.5265	0.6332

Saturation flow rates are calculated on the basis of reductions to the base saturation flow rate of 1,900 pc/hg/ln by using Equation 22-3. The lane utilization of the external approaches is obtained as shown above in Exhibit 22-68. Traffic pressure is calculated by using Equation 22-4. The left- and right-turn adjustment factors are estimated by using Equation 22-6 through Equation 22-9. These equations use an adjustment factor for travel path radius calculated by Equation 22-5. The remaining adjustment factors are calculated according to Chapter 18. The results of these calculations for the eastbound approaches are presented in Exhibit 22-69.

	EXT-TH	EXT-LT	INT-TH, INT-RT
Base saturation flow (s_0, in pc/hg/ln)	1,900	1,900	1,900
Number of lanes (N)	3	1	3
Lane width adjustment (f_w)	1.000	1.000	1.000
Heavy-vehicle adjustment (f_{hv})	0.909	1.000	0.909
Grade adjustment (f_g)	1.000	1.000	1.000
Parking adjustment (f_p)	1.000	1.000	1.000
Bus blockage adjustment (f_{bb})	1.000	1.000	1.000
Area type adjustment (f_a)	1.000	1.000	1.000
Lane utilization adjustment (f_{LU})	0.733	1.000	1.000
Left-turn adjustment (f_{LT})	1.000	0.934	1.000
Right-turn adjustment (f_{RT})	1.000	1.000	0.998
Left-turn pedestrian–bicycle adjustment (f_{Lpb})	1.000	1.000	1.000
Right-turn pedestrian–bicycle adjustment (f_{Rpb})	1.000	1.000	1.000
Turn radius adjustment for lane group (f_R)	1.000	0.934	0.985
Traffic pressure adjustment for lane group (f_v)	0.997	1.013	1.016
Adjusted saturation flow (s, in veh/hg/ln)	3786	1798	5253

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-69
Example Problem 2: Calculation of Saturation Flow Rates for Eastbound Approaches

Calculation of Common Green and Lost Time due to Downstream Queue

The duration of common green between various movements is calculated next. Exhibit 22-70 presents the beginning and ending of each phase at the two intersections and the calculations of common green between various movements at the two intersections.

Phase	Intersection I		Intersection II		
	Phase Begin	Phase End	Phase Begin	Phase End	
Phase 1	0	25	0	25	
Phase 2	30	90	30	65	
Phase 3	95	135	70	135	
Movement	**1st Green Time of Cycle**		**2nd Green Time of Cycle**		**Overlap**
	Begin	**End**	**Begin**	**End**	
EB EXT-TH	0	90			20
EB INT-TH	70	135			
WB EXT-TH	0	25	70	135	20
WB INT-TH	30	90			
SB RAMP	95	135			40
EB INT-TH	70	135			
NB RAMP	30	65			35
WB INT-TH	30	90			

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, TH = through, INT = internal, EXT = external.

Exhibit 22-70
Example Problem 2: Common Green Calculations

The next step involves the calculation of lost time due to downstream queues. First, the queue at the beginning of the upstream arterial phase and at the beginning of the upstream ramp phase must be calculated by using Equation 22-14 and Equation 22-15, respectively. These numbers are then subtracted from the internal link storage length to determine the storage at the beginning of the respective upstream phase. Exhibit 22-71 presents the calculation of the downstream queues followed by the calculation of the respective lost time due to those downstream queues.

Exhibit 22-71
Example Problem 2: Calculation of Lost Time due to Downstream Queues

	EB EXT-TH	SB-LT	WB EXT-TH	NB-LT
V_R or V_A (veh/h)	289	1066	229	1249
N_R or N_A	1	3	1	3
G_R or G_A (s)	40	90	35	95
G_D (s)	65	65	60	60
C (s)	140	140	140	140
CG_{UD} or CG_{RD} (s)	20	40	20	35
Queue length (Q_A or Q_R) (ft)	0.9	48.6	0.0	89.4
G_R or G_A (s)	90	40	95	35
C (s)	140	140	140	140
D_{QA} or D_{QR} (ft)	799	751	800	711
CG_{UD} or CG_{RD} (s)	20	40	20	35
Additional lost time ($L_{D\text{-}A}$ or $L_{D\text{-}R}$) (s)	0	0	0	0
Total lost time t'_L (s)	6	6	6	6
Effective green time g' (s)	89	39	94	34

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, LT = left turn, TH = through, EXT = external.

Queue Storage and Control Delay

The queue storage ratio is estimated as the ratio of the average maximum queue divided by the available queue storage by using Equation 31-91. Exhibit 22-72 presents the calculation of queue storage for all eastbound movements. Control delay for each movement is calculated according to Equation 18-47. Exhibit 22-73 provides the control delay for each eastbound movement of this interchange.

Exhibit 22-72
Example Problem 2: Queue Storage Ratio Calculations for Eastbound Movements

	EXT-TH	EXT-LT	INT-TH, INT-RT
Q_{bL} (ft)	0.0	0.0	0.0
v (veh/h/ln group)	1066	316	1282
s (veh/h/ln)	1262	1798	1751
g (s)	89	24	64
g/C	0.64	0.17	0.46
I	1.00	1.00	0.90
c (veh/h/ln group)	2407	308	2401
$X = v/c$	0.44	1.02	0.54
r_a (ft/s^2)	3.5	3.5	3.5
r_d (ft/s^2)	4.0	4.0	4.0
S_s (mi/h)	5	5	5
S_{pl} (mi/h)	40	40	40
S_a (mi/h)	39.96	39.96	39.96
d_a (s)	12.04	12.04	12.04
Rp	1.000	1.000	1.333
P	0.64	0.17	0.61
r (s)	51	116	76
t_f (s)	0.00	0.01	0.00
q (veh/s)	0.30	0.09	0.38
q_g (veh/s)	0.30	0.09	0.50
q_r (veh/s)	0.30	0.09	0.27
Q_1 (veh)	5.36	10.75	6.91
Q_2 (veh)	0.13	4.94	0.33
T	0.25	0.25	0.25
Q_{eo} (veh)	0.00	0.00	0.00
t_A	0	0	0
Q_e (veh)	0.00	0.00	0.00
Q_b (veh)	0.00	0.00	0.00
Q_3 (veh)	0.00	0.00	0.00
Q (veh)	5.49	15.69	7.24
L_h (ft)	25.02	25.00	25.02
L_a (ft)	800	200	800
R_Q	0.17	1.96	0.23

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external.

Exhibit 22-73
Example Problem 2: Control Delay for Eastbound Movements

	EXT-TH	INT-LT	INT-TH, INT-RT
g (s)	NA	24	64
g' (s)	89	NA	NA
g/C or g'/C	0.64	0.17	0.46
c (veh/h)	2407	308	2401
$X = v/c$	0.44	1.02	0.56
d_1 (s/veh)	12.9	58.0	18.81
k	0.5	0.5	0.5
d_2 (s/veh)	0.6	57.7	1.53
d_3 (s/veh)	0.0	0.0	0.0
k_{min}	0.04	0.04	0.04
u	0	0	0
t	0	0	0
d (s/veh)	13.5	115.7	20.34

Note: LT = left turn, RT = right turn, TH = through, INT = internal, EXT = external, NA = not applicable.

Results

This section discusses the estimation of delay experienced by each O-D movement and provides the resulting LOS. Delay for each O-D is estimated as a sum of the movement delays for each movement utilized by the O-D, as shown in Equation 22-1. Next, the v/c and queue storage ratios are checked. If either of these parameters exceeds 1, the LOS for all O-Ds that utilize that movement will be F. Exhibit 22-74 presents the resulting delay, v/c, and R_Q for each O-D movement. As shown, O-D Movement F has a v/c and R_Q ratio greater than 1, resulting in a LOS of F.

Exhibit 22-74
Example Problem 2: O-D Movement LOS

O-D Movement	Delay (s)	$v/c > 1$?	$R_Q > 1$?	LOS
A	78.9	No	No	D
B	55.7	No	No	D
C	41.1	No	No	C
D	70.0	No	No	D
E	33.9	No	No	C
F	115.7	Yes	Yes	F
G	61.6	No	No	D
H	40.0	No	No	C
I	33.9	No	No	C
J	40.0	No	No	C

EXAMPLE PROBLEM 3: OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

The Interchange

An interchange is to be built at the junction of I-83 (NB/SB) and Archer Road (EB/WB) in an urban area. The interchange type selection methodology is used.

The Question

Which interchange type is likely to operate better under the given demands?

The Facts

This interchange will have two-lane approaches with single left-turn lanes on the arterial approaches. Freeway ramps will consist of two-lane approaches with channelized right turns in addition to the main ramp lanes. Default saturation flow rates for use in the interchange type selection analysis are given in Exhibit 22-28. The O-Ds of traffic through this interchange are shown in Exhibit 22-75.

Exhibit 22-75
Example Problem 3: O-D Demand Information for the Interchange

O-D Movement	Volume (veh/h)
A	400
B	350
C	400
D	550
E	150
F	200
G	225
H	185
I	600
J	800
K	2,500
L	3,200
M	0
N	10

Outline of Solution

Mapping O-D Flows into Interchange Movements

The primary objective of this example is to compare up to eight interchange types against a given set of design volumes. The first step is to convert these O-D flows into movement flows through the signalized interchange. The interchange type methodology uses the standard NEMA numbering sequence for interchange phasing, and Exhibit 22-29 demonstrates which O-Ds make up each NEMA phase at the eight interchange types. Exhibit 22-76 shows the corresponding volumes for this example on the basis of the O-Ds from Exhibit 22-75. Since this interchange has channelized right turns, Exhibit 22-77 shows only the NEMA phasing volumes utilizing the signals.

Exhibit 22-76
Example Problem 3: NEMA Flows (veh/h) for the Interchange

Interchange Type	NEMA Phase Movement Number							
	1	2	3	4	5	6	7	8
SPUI	185	800	400	400	150	1,025	560	350
TUDI /CUDI	185	950	--	960	160	1,210	--	750
CDI (I)	185	950	--	960	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	960	--	1,385	--	--
Parclo A-4Q (II)	--	1,310	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	960	200	1,385	--	--
Parclo A-2Q (II)	225	1,310	--	--	--	985	--	750
Parclo B-4Q (I)	185	950	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	950	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Note: (I) and (II) indicate the intersections within the interchange type; -- indicates that the movement does not exist in this interchange type.

Exhibit 22-77
Example Problem 3: NEMA Flows for the Interchange Without Channelized Right Turns

Interchange Type	NEMA Phase Movement Number							
	1	2	3	4	5	6	7	8
SPUI	185	600	400	0	150	1,025	560	350
TUDI /CUDI	185	750	--	560	160	1,210	--	750
CDI (I)	185	750	--	560	--	1,200	--	--
CDI (II)	--	1,150	--	--	160	1,210	--	750
Parclo A-4Q (I)	--	750	--	560	--	1,385	--	--
Parclo A-4Q (II)	--	1,150	--	--	--	985	--	750
Parclo A-2Q (I)	--	750	--	560	200	1,385	--	--
Parclo A-2Q (II)	225	1,150	--	--	--	985	--	750
Parclo B-4Q (I)	185	750	--	--	--	1,200	--	--
Parclo B-4Q (II)	--	1,150	--	--	160	1,210	--	--
Parclo B-2Q (I)	185	750	--	--	--	1,200	--	400
Parclo B-2Q (II)	--	1,150	--	350	160	1,210	--	--

Computation of Critical Flow Ratios

Comparison between the eight intersection types begins with computation of the critical flow ratio at each interchange type. The first intersection type to be calculated is the SPUI by using Equation 22-26. On the basis of the default saturation flow rate for a SPUI and the values for the NEMA phases, Exhibit 22-78 shows the output from these calculations for a SPUI.

Exhibit 22-78
Example Problem 3: SPUI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements, *A*	0.368	0.306
Critical flow ratio for the ramp movements, *R*	0.350	0.156
Sum of critical flow ratios, Y_c	0.718	0.462

The TUDI critical flow ratios are calculated by using Equation 22-27. Exhibit 22-79 shows these calculations for a 300-ft distance between the two TUDI intersections.

Exhibit 22-79
Example Problem 3: TUDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Effective flow ratio for concurrent phase when dictated by travel time, y_t	0.070	0.070
Effective flow ratio for concurrent Phase 3, y_3	0.070	0.070
Effective flow ratio for concurrent Phase 7, y_7	0.070	0.070
Critical flow ratio for the arterial movements, *A*	0.461	0.294
Critical flow ratio for the ramp movements, *R*	0.474	0.315
Sum of critical flow ratios, Y_c	0.935	0.609

The CUDI critical flow ratios are calculated by using Equation 22-28. Exhibit 22-80 shows these calculations for a CUDI with the given O-D flows.

Exhibit 22-80
Example Problem 3: CUDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Flow ratio for Phase 2 with consideration of prepositioning, y_2	0.264	0.208
Flow ratio for Phase 6 with consideration of prepositioning, y_6	0.208	0.208
Critical flow ratio for the arterial movements, A	0.373	0.332
Critical flow ratio for the ramp movements, R	0.267	0.156
Sum of critical flow ratios, Y_c	0.640	0.488

The CDI, Parclo A-4Q, Parclo A-2Q, Parclo B-4Q, and Parclo B-2Q all use separate controllers. For these interchanges the critical flow ratios are calculated for each intersection, and then the maximum is taken for the overall interchange critical flow ratio. These numbers are all calculated by using Equation 22-29 and the default saturation flows. Exhibit 22-81 through Exhibit 22-85 show the calculations for these interchanges utilizing two controllers.

Exhibit 22-81
Example Problem 3: CDI Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.655	0.498
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.118
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.486
Maximum sum of critical flow ratios, Y_c	0.655	0.498

Exhibit 22-82
Example Problem 3: Parclo A-4Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.385	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.282
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.667	0.615
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.364	0.333
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.208	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.572	0.444
Maximum sum of critical flow ratios, Y_c	0.667	0.615

Exhibit 22-83
Example Problem 3: Parclo A-2Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.502	0.451
Critical flow ratio for the ramp movements at Intersection I, R_I	0.282	0.165
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.784	0.616
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.452
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.221	0.111
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.651	0.563
Maximum sum of critical flow ratios, Y_c	0.784	0.616

Exhibit 22-84
Example Problem 3: Parclo B-4Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.000	0.000
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.373	0.333
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.000	0.000
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.430	0.368
Maximum sum of critical flow ratios, Y_c	0.430	0.368

Exhibit 22-85
Example Problem 3: Parclo B-2Q Critical Flow Ratio Calculations

	Signalized Right Turns	Channelized Right Turns
Critical flow ratio for the arterial movements at Intersection I, A_I	0.373	0.333
Critical flow ratio for the ramp movements at Intersection I, R_I	0.111	0.111
Sum of critical flow ratios at Intersection I, $Y_{c,I}$	0.484	0.444
Critical flow ratio for the arterial movements at Intersection II, A_{II}	0.430	0.368
Critical flow ratio for the ramp movements at Intersection II, R_{II}	0.103	0.103
Sum of critical flow ratios at Intersection II, $Y_{c,II}$	0.533	0.471
Maximum sum of critical flow ratios, Y_c	0.533	0.471

Estimation of Interchange Delay

Estimation of interchange delay is the final step when interchange types are compared. On the basis of the critical flow ratios calculated previously, Exhibit 22-36 can be used to calculate the delay at the eight interchange types. Exhibit 22-86 shows the solutions to these calculations.

Exhibit 22-86
Example Problem 3: Interchange Delay for the Eight Interchange Types

Intersection Type	Interchange Delay d_I (s) Right Turns Signalized	Interchange Delay d_I (s) Right Turns Free or YIELD-controlled
SPUI	62.9	22.0
TUDI	217.7	33.3
CUDI	35.9	27.4
CDI	26.6	21.7
Parclo A-4Q	26.2	21.6
Parclo A-2Q	47.4	29.0
Parclo B-4Q	11.9	11.3
Parclo B-2Q	30.7	29.0

Results

As demonstrated by Exhibit 22-86, a Parclo B-4Q would be the best interchange type to select in terms of operational performance for the given O-D flows at this interchange. For the final interchange type selection, however, additional criteria should be considered, including those related to economic, environmental, and land use concerns.

5. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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2. Elefteriadou, L., A. Elias, C. Fang, C. Lu, L. Xie, and B. Martin. *Validation and Enhancement of the Highway Capacity Manual's Interchange Ramp Terminal Methodology.* Final Report, National Cooperative Highway Research Program Project 3-60A. University of Florida, Gainesville, 2009.

3. Messer, C. J., and J. A. Bonneson. *Capacity of Interchange Ramp Terminals.* Final Report, National Cooperative Highway Research Program Project 3-47. Texas A&M Research Foundation, College Station, April 1997.

4. Bonneson, J., K. Zimmerman, and M. Jacobson. *Review and Evaluation of Interchange Ramp Design Considerations for Facilities Without Frontage Roads.* Research Report 0-4538-1. Cooperative Research Program, Texas Transportation Institute, Texas A&M University System, College Station, 2004.

5. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*, 5th ed. Washington, D.C., 2004.

CHAPTER 23
OFF-STREET PEDESTRIAN AND BICYCLE FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

Off-street pedestrian and bicycle facilities are facilities that (*a*) are used only by nonmotorized modes and (*b*) are not considered part of an urban street or transit facility. The second part of this definition excludes facilities located directly along an urban street (e.g., bicycle lanes or sidewalks). In general, off-street facilities include those for which the characteristics of motor vehicle traffic do not play a strong role in determining the quality of service from the perspective of bicyclists and pedestrians. Thus, a shared-use path only 10 ft from a roadway but separated by a sound barrier may be considered an off-street facility, whereas a sidepath with a 10-ft planted buffer separating it from the roadway would generally be considered an on-street facility.

In general, facilities located within approximately 35 ft of an urban street are not considered off-street, although the precise definition of "off-street" varies by facility as described earlier. These types of pedestrian and bicycle facilities are covered in Chapter 16, Urban Street Facilities, and Chapter 17, Urban Street Segments. The definition also excludes crosswalks and queuing areas; these are addressed in each of the chapters on intersections (Chapters 18–21). Pedestrian components of transit facilities are addressed in the *Transit Capacity and Quality of Service Manual* (*1*). The 35-ft threshold is based on studies of pedestrian and bicycle facilities (*2–4*) in which it was found that motor vehicle traffic influenced pedestrian and bicycle quality of service on facilities located within at least this distance of the roadway.

Chapter 23, Off-Street Pedestrian and Bicycle Facilities, provides capacity and level-of-service (LOS) estimation procedures for the types of facilities shown below. Examples of each of the following facility types can be found in Chapter 3, Modal Characteristics:

- Walkways: paved paths, ramps, and plazas that are generally located more than 35 ft from an urban street as well as streets reserved for pedestrian traffic on a full- or part-time basis;
- Stairways: staircases that are part of a longer pedestrian facility;
- Shared-use paths: paths physically separated from highway traffic for the use of pedestrians, bicyclists, runners, inline skaters, and other users of nonmotorized modes; and
- Exclusive off-street bicycle paths: paths physically separated from highway traffic for the exclusive use of bicycles.

Pedestrian capacity concepts are the same across facility types (i.e., exclusive off-street facilities, on-street facilities, and transit facilities). However, LOS thresholds for transit facilities allow higher levels of crowding for a given LOS than do the thresholds for nontransit facilities.

Off-street facilities are those generally located more than 35 ft from a roadway, although the exact distance may vary on the basis of the local context.

ANALYSIS BOUNDARIES

The analysis of off-street pedestrian and bicycle facilities occurs at the segment level. A segment ends and a new segment begins when any of the following occur:

- There is a street crossing;
- The width of the facility changes significantly;
- There is an intersection with another exclusive pedestrian or bicycle facility, where user volumes change significantly or cross flows are created; or
- The type of facility changes (e.g., where a walkway becomes a stairway).

LOS CRITERIA

The LOS thresholds defined for each of the off-street pedestrian and bicycle facilities are presented in this section. Three types of service measures are defined:

- For pedestrians on exclusive pedestrian facilities, *pedestrian space* (square feet per pedestrian);
- For pedestrians on facilities shared by pedestrians and bicycles, the number of *bicycle meeting and passing events per hour*; and
- For bicycles on both shared-use and exclusive paths, a *bicycle LOS score* incorporating meetings per minute, active passings per minute, presence of a centerline, path width, and delayed passings.

Exhibit 23-1 through Exhibit 23-5 provide five LOS tables: four for pedestrian facilities and one for bicycle facilities. As described in Chapter 4, Traffic Flow and Capacity Concepts, pedestrian flow rates and speeds are directly related to the average space occupied by a pedestrian. These values are given for reference in the space-based LOS tables along with the corresponding range of volume-to-capacity (v/c) ratios; however, the actual LOS in those tables are based on space per pedestrian.

LOS does not reflect whether a facility complies with the ADA or other standards.

The LOS thresholds are based on user perception research where available and in other cases on expert judgment. LOS does not reflect whether a facility complies with the Americans with Disabilities Act (ADA) or other standards.

Walkways

The walkway LOS tables apply to paved pedestrian paths, pedestrian zones (exclusive pedestrian streets), walkways and ramps with up to a 5% grade, and pedestrian walking zones through plaza areas. Exhibit 23-1 applies when pedestrian flow along the facility is random. Exhibit 23-2 applies when platoons of pedestrians form along the facility, for example, when a signalized crosswalk is located at one end of the portion of the facility being analyzed.

Cross flows occur at the intersection of two approximately perpendicular pedestrian streams (e.g., where two walkways intersect or at a building entrance). Because of the increased number of conflicts that occur between pedestrians, walkway capacity is lower in a cross-flow situation than along other parts of the walkway. In cross-flow locations, the LOS E–F threshold is 13 ft^2/p, as indicated in the notes for Exhibit 23-1 and Exhibit 23-2.

Exhibit 23-1
Average Flow LOS Criteria for Walkways

LOS	Average Space (ft^2/p)	Related Measures: Flow Rate (p/min/ft)[a]	Average Speed (ft/s)	v/c Ratio[b]	Comments
A	>60	≤5	>4.25	≤0.21	Ability to move in desired path, no need to alter movements
B	>40–60	>5–7	>4.17–4.25	>0.21–0.31	Occasional need to adjust path to avoid conflicts
C	>24–40	>7–10	>4.00–4.17	>0.31–0.44	Frequent need to adjust path to avoid conflicts
D	>15–24	>10–15	>3.75–4.00	>0.44–0.65	Speed and ability to pass slower pedestrians restricted
E	>8–15[c]	>15–23	>2.50–3.75	>0.65–1.00	Speed restricted, very limited ability to pass slower pedestrians
F	≤8[c]	Variable	≤2.50	Variable	Speeds severely restricted, frequent contact with other users

Notes: Exhibit 23-1 does not apply to walkways with steep grades (>5%). See the Special Cases section for further discussion.
[a] Pedestrians per minute per foot of walkway width.
[b] v/c ratio = flow rate/23. LOS is based on average space per pedestrian.
[c] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Exhibit 23-2
Platoon-Adjusted LOS Criteria for Walkways

LOS	Average Space (ft^2/p)	Related Measure: Flow Rate[a] (p/min/ft)[b]	Comments
A	>530	≤0.5	Ability to move in desired path, no need to alter movements
B	>90–530	>0.5–3	Occasional need to adjust path to avoid conflicts
C	>40–90	>3–6	Frequent need to adjust path to avoid conflicts
D	>23–40	>6–11	Speed and ability to pass slower pedestrians restricted
E	>11–23[c]	>11–18	Speed restricted, very limited ability to pass slower pedestrians
F	≤11[c]	>18	Speeds severely restricted, frequent contact with other users

Notes: [a] Rates in the table represent average flow rates over a 5-min period. Flow rate is directly related to space; however, LOS is based on average space per pedestrian.
[b] Pedestrians per minute per foot of walkway width.
[c] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Stairways

Exhibit 23-3 provides the LOS criteria for stairways.

Exhibit 23-3
LOS Criteria for Stairways

LOS	Average Space (ft^2/p)	Related Measures: Flow Rate (p/min/ft)[a]	v/c Ratio[b]	Comments
A	>20	≤5	≤ 0.33	No need to alter movements
B	>17–20	>5–6	>0.33–0.41	Occasional need to adjust path to avoid conflicts
C	>12–17	>6–8	>0.41–0.53	Frequent need to adjust path to avoid conflicts
D	>8–12	>8–11	>0.53–0.73	Limited ability to pass slower pedestrians
E	>5–8	>11–15	>0.73–1.00	Very limited ability to pass slower pedestrians
F	≤5	Variable	Variable	Speeds severely restricted, frequent contact with other users

Notes: [a] Pedestrians per minute per foot of walkway width.
[b] v/c ratio = flow rate/15. LOS is based on average space per pedestrian.

Pedestrians on Shared-Use Paths

Exhibit 23-4 shows LOS criteria for paths shared between pedestrians and bicycles.

Exhibit 23-4
Pedestrian LOS Criteria for Shared-Use Paths

LOS	Weighted Event Rate/h	Related Measure Bicycle Service Flow Rate per Direction (bicycles/h)	Comments
A	≤38	≤28	Optimum conditions, conflicts with bicycles rare
B	>38–60	>28–44	Good conditions, few conflicts with bicycles
C	>60–103	>44–75	Difficult to walk two abreast
D	>103–144	>75–105	Frequent conflicts with cyclists
E	>144–180	>105–131	Conflicts with cyclists frequent and disruptive
F	>180	>131	Significant user conflicts, diminished experience

Notes: An "event" is a bicycle meeting or passing a pedestrian.
Bicycle service volumes are shown for reference and are based on a 50/50 directional split of bicycles; LOS is based on number of events per hour and applies to any directional split.

Exclusive and Shared Bicycle Facilities

Exhibit 23-5 provides LOS criteria for bicyclists on both shared-use and exclusive off-street paths.

Exhibit 23-5
LOS Criteria for Bicycles on Shared-Use and Exclusive Paths

LOS	Bicycle LOS Score	Comments
A	>4.0	Optimum conditions, ample ability to absorb more riders
B	>3.5–4.0	Good conditions, some ability to absorb more riders
C	>3.0–3.5	Meets current demand, marginal ability to absorb more riders
D	>2.5–3.0	Many conflicts, some reduction in bicycle travel speed
E	>2.0–2.5	Very crowded, with significantly reduced bicycle travel speed
F	≤2.0	Significant user conflicts and diminished experience

REQUIRED INPUT DATA

The input data required to perform an analysis differ depending on the type of facility and user being analyzed. Exhibit 23-6 shows the required input data for each of the facility types addressed in this chapter.

Exhibit 23-6
Required Input Data by Exclusive Pedestrian and Bicycle Facility Type

Pedestrian/Bicycle Facility	Required Input Data	Symbol
Walkways, stairways	Effective walkway width	E_w
	Peak 15-min pedestrian volume	v_{15}
Pedestrians on shared-use paths	Directional hourly bicycle volumes	Q_b
	Mean pedestrian speed	S_p
	Mean bicycle speed	S_b
Bicycles on shared-use and exclusive paths	Directional hourly path volumes	Q_T
	Path mode split by user group	p_i
	Path peak hour factor	PHF
	Mean and standard deviation of speed by user group	μ_i, σ_i
	Path width	--
	Presence of centerline on path	CL
	Proportion of users blocking two lanes by user group (three- and four-lane paths only)	P_b

SCOPE OF THE METHODOLOGY

Methodologies are described for evaluating the LOS of pedestrian and bicycle facilities that are separate from, and unaffected by, motor vehicle traffic. Other chapters in Volumes 2 and 3 provide methodologies for determining pedestrian and bicycle LOS on roadway system elements with motor vehicle traffic. The procedures may be applied in an approximate manner to pedestrian zones (exclusive pedestrian streets), plazas, and ramps with grades exceeding 5%, as described later in the Special Cases section.

The analysis methodologies are based solely on facility characteristics and do not consider external factors that may also affect quality of service, such as weather, landscaping, adjacent land uses, and lighting conditions, which may also affect users' perceptions of a facility.

Much of the material in this chapter is the result of research sponsored by the Federal Highway Administration (*5–7*). Both commuter and recreational bicyclists were included in the off-street bicycle path research (*7*).

LIMITATIONS OF THE METHODOLOGY

In this chapter each of the facilities is treated from the point of view of pedestrians or bicyclists. Procedures for assessing the impact of pedestrians and bicyclists on other facility users (e.g., inline skaters) are not considered. Additional information on other users may be found elsewhere (*8*). The methodology does not address LOS for pedestrians with disabilities, including vision or mobility impairments. The reader is encouraged to consult material published by the United States Access Board to ensure compliance with the ADA.

The analysis methodologies presented here do not consider the continuity of walkways, bikeways, and shared-use paths in determining the LOS. Facilities that are interrupted with frequent roadway crossings will provide lower capacities and travel speeds than facilities with long, uninterrupted stretches. In addition, roadway crossings, especially crossings of high-volume or high-speed facilities, may negatively affect the pedestrian and bicycle environment and user perceptions of quality of service. However, the methodologies described here only consider discrete, uninterrupted facilities and do not assess the impact of intersections with other facilities.

The methodology does not address the impact of roadway crossings on the LOS of off-street paths.

Pedestrian Facilities

The capacity of pedestrian facilities is based on research conducted on constrained facilities (e.g., bridges and underground passageways), where there is no opportunity for pedestrians to walk outside the designated area. Off-street pedestrian facilities, in contrast, typically have no barriers keeping pedestrians to the designated path. As a result, these facilities reach effective failure (i.e., pedestrian spillover) at densities less than their capacity. For this reason, in combination with considerations of general pedestrian comfort, off-street walkways are desirably designed to achieve LOS C or better, based on pedestrian space, rather than for capacity conditions. The methodologies are generally appropriate regardless of the type of surface used for the pedestrian facility.

Where the opportunity exists, pedestrians will spill over the edges of a walkway at densities below capacity.

Exclusive Bicycle Facilities

The methodology for exclusive bicycle facilities is based on research conducted only on paved surfaces and may not be applicable to soft surfaces such as gravel, dirt, or wood chips.

The exclusive bicycle facility methodology may not be applicable to facilities with soft surfaces.

The pedestrian shared-use-path methodology does not account for the effects of nonbicyclist users of the path on pedestrian LOS.

Shared-Use Paths

The methodology for shared-use paths does not account for the effect on pedestrian LOS of path width or the impact of meeting and passing events. No credible data were found on fixed objects and their effects on users of these types of facilities. The methodology also does not account for the effect of nonbicyclist users of the path (e.g., skateboarders, inline skaters) on pedestrians. However, it is expected that pedestrians will often encounter these users on shared-use paths and that because of their higher speeds, these users can have a negative effect on pedestrian LOS.

The methodology for bicycle LOS on shared-use paths incorporates the effects of five user groups: bicyclists, pedestrians, runners, inline skaters, and child bicyclists. However, several path user groups that may be a part of the mix on some trails are not incorporated, including push scooter users, wheelchair users, equestrians, cross-country skiers, and users of electric vehicles. The methodology is based on research conducted only on paved surfaces and may not be applicable to soft surfaces such as gravel, dirt, or wood chips. The methodology is not applicable for paths wider than 20 ft. This methodology was developed from data collected on two-way paths but may be applied to one-way paths by setting opposing volumes equal to zero.

Some shared-use paths are signed or striped, or both, to segregate pedestrian and bicycle traffic. The research that developed the shared-use-path methodology did not address those kinds of paths; guidance on such paths may be found in the Special Cases section.

2. METHODOLOGY

OVERVIEW

Off-street pedestrian and bicycle facilities serve only nonmotorized traffic and are separated from motor vehicle traffic to the extent that such traffic does not affect their quality of service. Thus, although sidewalks primarily serve only pedestrians, they are not addressed in this chapter—the quality of service afforded to pedestrians on sidewalks depends in part on the presence and characteristics of the adjacent motor vehicle traffic.

Sidewalks and bicycle facilities along urban streets are addressed in Chapter 17, Urban Street Segments.

Bicycle facilities on multilane and two-lane highways are addressed in Chapters 14 and 15, respectively.

Procedures for estimating LOS are separated into three main categories: exclusive pedestrian facilities, exclusive bicycle facilities, and shared pedestrian and bicycle facilities. Separate methodologies are provided to assess pedestrian and bicycle LOS on shared facilities.

There are three general categories of exclusive pedestrian facilities: walkways, cross-flow areas, and stairways. The LOS thresholds for each category are different, but all are based on the concept of space per pedestrian, which is a measure of pedestrian comfort and mobility. Exhibit 23-7 illustrates the steps taken to determine the LOS of exclusive off-street pedestrian facilities.

Exhibit 23-7
Flowchart for Analysis of Exclusive Off-Street Pedestrian Facilities

Step 1: Determine the effective walkway width

↓

Step 2: Calculate the pedestrian flow rate

↓

Step 3: Calculate the average pedestrian space

↓

Step 4: Determine LOS
Exhibit 23-1 (walkways without platooning)
Exhibit 23-2 (walkways with platooning)
Exhibit 23-3 (stairways)

↓

Step 5: Calculate the volume-to-capacity ratio

LOS for pedestrians on shared off-street bicycle and pedestrian facilities is based on the number of events during which a pedestrian either meets an oncoming bicyclist or is passed by a bicyclist. As the number of events increases, the pedestrian LOS decreases because of reduced comfort. Exhibit 23-8 shows the steps taken to determine shared-facility LOS.

Exhibit 23-8
Flowchart for Analysis of Pedestrian LOS on Shared Off-Street Facilities

Step 1: Gather input data
Hourly pedestrian and bicycle demands by direction
Bicycle and pedestrian speeds

↓

Step 2: Calculate the number of bicycle passing and meeting events

↓

Step 3: Determine LOS
Exhibit 23-4

Bicycle LOS on exclusive and shared-use off-street bicycle facilities is based on user perceptions of how the LOS of shared-use paths changes according to several different factors. These factors are combined into a single bicycle LOS score. LOS thresholds relate to a specific range of LOS score values. Exhibit 23-9 shows the steps taken to determine the LOS of off-street bicycle facilities.

Exhibit 23-9
Flowchart for Analysis of Bicycle LOS on Off-Street Facilities

Step 1: Gather input data
For each modal user (pedestrians, bicyclists, inline skaters, runners, and child bicyclists):
Hourly demands by direction, speeds, and mode split
For the facility:
Path width, centerline presence

↓

Step 2: Calculate active passings per minute

↓

Step 3: Calculate meetings per minute

↓

Step 4: Determine the number of lanes

↓

Step 5: Calculate the probability of delayed passing

↓

Step 6: Calculate delayed passings per minute

↓

Step 7: Determine LOS
Exhibit 23-5

↓

Step 8: Adjust LOS for low-volume paths

EXCLUSIVE OFF-STREET PEDESTRIAN FACILITIES

Step 1: Determine Effective Walkway Width

Walkways and Cross-Flow Areas

Effective walkway width is the portion of a walkway that can be used effectively by pedestrians. Various types of obstructions and linear features, discussed below, reduce the walkway area that can be effectively used by pedestrians. The effective walkway width at a given point along the walkway is computed as follows:

$$W_E = W_T - W_O$$

Equation 23-1

where

W_E = effective walkway width (ft),

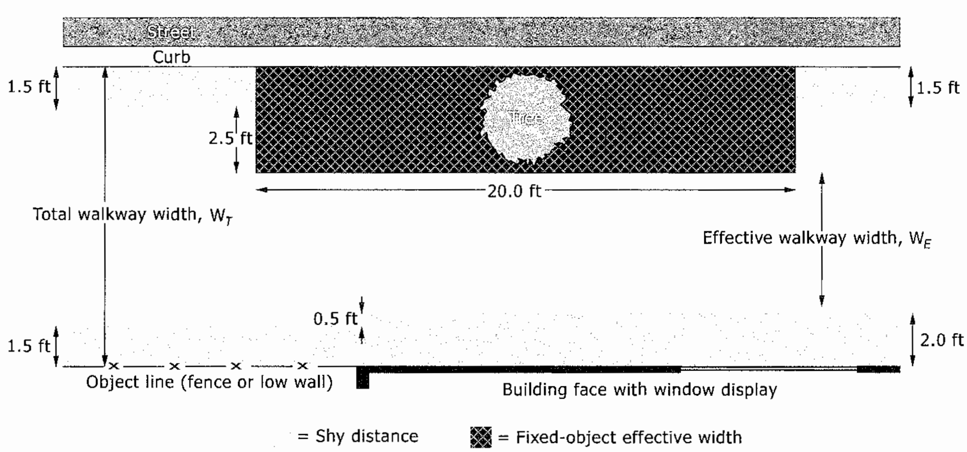
W_T = total walkway width at a given point along walkway (ft), and

W_O = sum of fixed-object effective widths and linear-feature shy distances at a given point along walkway (ft).

Shy distance is a buffer that pedestrians leave between themselves and linear objects along a walkway, such as curbs and building faces.

Exhibit 23-10 illustrates a portion of a sidewalk or walkway. The general concepts shown are applicable both to sidewalks along urban streets and to exclusive off-street paths not located adjacent to a street. Linear features such as the street curb, the low wall, and the building face each have associated shy distances. The *shy distance* is the buffer that pedestrians give themselves to avoid accidentally stepping off the curb, brushing against a building face, or getting too close to other pedestrians standing under awnings or window shopping. Fixed objects, such as the tree, have effective widths associated with them. The *fixed-object effective width* includes the object's physical width, any functionally unusable space (e.g., the space between a parking meter and the curb or the space in front of a bench occupied by people's legs and belongings), and the buffer given the object by pedestrians.

Exhibit 23-10
Width Adjustments for Fixed Obstacles



The concept of effective width applies to both on-street and off-street facilities. Because of the proximity of the street in Exhibit 23-10, the sidewalk here would be considered an on-street pedestrian facility.

The street is shown so that all factors that can influence the effective width of walkways can be depicted in one place.

Exhibit 23-10 also shows that the effective width of a fixed object extends over an effective length that is considerably longer than the object's physical length. The effective length represents the portion of the walkway that is functionally unusable because pedestrians need to move to one side ahead of time to get around a fixed object. The effective length of a fixed object is assumed to be five times the object's effective width.

Typically, a walkway operational analysis evaluates the portion of the walkway with the narrowest effective width, since this section forms the constraint on pedestrian flow. A design analysis identifies the minimum effective width that must be maintained along the length of the walkway to avoid pedestrian queuing or spillover.

Exhibit 23-11 gives the effective widths of a variety of typical fixed objects found along on- and off-street pedestrian facilities. The values in Exhibit 23-11 can be used when specific walkway configurations are not available.

Exhibit 23-11
Typical Fixed-Object Effective Widths

Fixed Object	Effective Width (ft)
Street Furniture	
Light pole	2.5–3.5
Traffic signal poles and boxes	3.0–4.0
Fire alarm boxes	2.5–3.5
Fire hydrants	2.5–3.0
Traffic signs	2.0–2.5
Parking meters	2.0
Mail boxes (1.7 ft x 1.7 ft)	3.2–3.7
Telephone booths (2.7 ft x 2.7 ft)	4.0
Trash cans (1.8 ft diameter)	3.0
Benches	5.0
Bus shelters (on sidewalk)	6.0–7.0
Public Underground Access	
Subway stairs	5.5–7.0
Subway ventilation gratings (raised)	6.0+
Transformer vault ventilation gratings (raised)	6.0+
Landscaping	
Trees	3.0–4.0
Planter boxes	5.0
Commercial Uses	
Newsstands	4.0–13.0
Vending stands	Variable
Advertising and store displays	Variable
Sidewalk cafés (two rows of tables)	7.0
Building Protrusions	
Columns	2.5–3.0
Stoops	2.0–6.0
Cellar doors	5.0–7.0
Standpipe connections	1.0
Awning poles	2.5
Truck docks (trucks protruding)	Variable
Garage entrance/exit	Variable
Driveways	Variable

Source: Pushkarev and Zupan (*9*).

See Exhibit 23-10 for shy distances associated with curbs and building faces.

Stairways

Pedestrians tend to walk in lines or lanes on stairways; thus, meaningful increases in capacity are related to the number of pedestrian lanes available.

A stairway's capacity is largely affected by its width. Unlike walking on a level surface, traversing stairs tends to make people walk in lines or lanes. The width of a stairway determines both the number of distinct lines that can traverse the stair and the side-to-side spacing between them, affecting both the ability of faster pedestrians to pass slower-moving pedestrians and the level of interference between adjacent lines of people. The consequence is that meaningful increases in capacity are not linearly proportional to the width but occur in increments of about 30 in. (*1*).

Small reverse flows on stairways should be assumed to use one pedestrian lane (30 in.) of width.

On stairways (in contrast to walkways), a minor pedestrian flow in the opposing direction can result in reduced capacity disproportionate to the magnitude of the reverse flow. As a result, a small reverse flow should be assumed to occupy one pedestrian lane or 30 in. of the stair's width. For a stairway with an effective width of 60 in. (5 ft), a small reverse flow could consume half its capacity (*1*). The allowance for small reverse flows, when used, is included as part of the W_O term in Equation 23-1.

Step 2: Calculate Pedestrian Flow Rate

Walkways and Cross-Flow Areas

Hourly pedestrian demands is used as an input to the analysis. Consistent with the general analysis procedures used throughout the HCM, hourly demand is usually converted into peak 15-min flows, so that LOS is based on the busiest 15 consecutive minutes during an hour:

Equation 23-2

$$v_{15} = \frac{v_h}{4 \times PHF}$$

where

v_{15} = pedestrian flow rate during peak 15 min (p/h),

v_h = pedestrian demand during analysis hour (p/h), and

PHF = peak hour factor.

However, if peak-15-min pedestrian volumes are available, the highest 15-min volume can be used directly without the application of a peak hour factor.

Next, the peak 15-min flow is converted into a unit flow rate (pedestrians per minute per foot of effective path width):

Equation 23-3

$$v_p = \frac{v_{15}}{15 \times W_E}$$

where v_p is pedestrian flow per unit width (p/ft/min) and all other variables are as previously defined.

Stairways

Critical pedestrian flows on stairs occur in the up direction.

Because pedestrians use more energy to ascend stairs than to descend them, lower flow rates typically occur in the ascending direction. For this reason, when stairs serve both directions simultaneously or when the same stairway will be used primarily in the up direction during some time periods and primarily in the down direction during other time periods, the upward flow rate should be used for analysis and design (*1*). The calculation of pedestrian flow rate for stairways is otherwise the same as that described for walkways and cross-flow areas.

Space = $\frac{1}{Density}$

Step 3: Calculate Average Pedestrian Space

The service measure for walkways is *pedestrian space,* the inverse of density. Pedestrian space can be directly observed in the field by measuring a sample area of the facility and determining the maximum number of pedestrians at a given time in that area. The pedestrian unit flow rate is related to pedestrian space and speed:

Equation 23-4

$$A_p = \frac{S_p}{v_p}$$

where

A_p = pedestrian space (ft^2/p),

S_p = pedestrian speed (ft/min), and

v_p = pedestrian flow per unit width (p/ft/min).

Step 4: Determine LOS

Walkways with Random Pedestrian Flow

Where pedestrian flow on the path is not influenced by platooning (see next subsection), Exhibit 23-1 should be used to determine pedestrian LOS.

Ramps with grades of 5% or less can be treated as walkways for the purpose of determining LOS.

Research (*9–11*) has shown that pedestrian speeds on ramps with grades up to 5% are not significantly different from speeds on level walkways but that speeds decrease at higher grades. Therefore, the walkway LOS values are also applicable to ramps with grades of 5% or less. Ramps with steeper grades are discussed later in this chapter in the Special Cases section. The walkway LOS values can also be adapted to pedestrian plazas and pedestrian zones (exclusive pedestrian streets), as discussed in the Special Cases section.

Walkways with Platoon Flow

Platooning on walkways.

It is important for the analyst to determine whether platooning alters the underlying assumptions of random flow in the LOS calculation. Platoons can arise, for example, if entry to a walkway segment is controlled by a traffic signal at a street crossing or if pedestrians arrive at intervals on transit vehicles.

Where platooning occurs, the pedestrian flow is concentrated over short time periods rather than being distributed evenly throughout the peak 15-min analysis period. The available space for the typical pedestrian under these circumstances is much more constrained than the average space available with random arrival would indicate. There is no strict definition for what constitutes platooning rather than random flow; observations of local conditions and engineering judgment should be used to determine the most relevant design criteria (i.e., platoons versus random flow).

If platooning occurs, Exhibit 23-2 should be used to determine LOS. Research (*9*) indicates that impeded flow starts at 530 ft^2/p, which is equivalent to a flow rate of 0.5 p/min/ft. This value is used as the LOS A–B threshold. The same research shows that jammed flow in platoons starts at 11 ft^2/p, which is equivalent to 18 p/min/ft. This value is used as the LOS E–F threshold.

Cross-Flow Areas

Cross-flow LOS thresholds are identical to those for walkways, except for the LOS E–F threshold.

A cross flow is a pedestrian flow that is approximately perpendicular to and crosses another pedestrian stream, for example, at the intersection of two walkways or at a building entrance. In general, the lesser of the two flows is referred to as the cross-flow condition. The same procedure used to estimate walkway space is used to analyze pedestrian facilities with cross flows. As shown in the footnotes to Exhibit 23-1 and Exhibit 23-2, the LOS E threshold (i.e., capacity) in cross-flow situations occurs at a lower density than that for walkways without cross flows (*12*).

Stairways

Research (*13*) has developed LOS thresholds based on the Institute of Transportation Engineers stairway standards, which provide the space and flow values given in Exhibit 23-3. As with walkways, stairway LOS is described by the service measure of pedestrian space, expressed as square feet per pedestrian.

Step 5: Calculate Volume-to-Capacity Ratio

The volume-to-capacity (*v/c*) ratio can be computed by using the following values of capacity for various exclusive pedestrian facilities:

- Walkways with random flow: 23 p/min/ft,
- Walkways with platoon flow (average over 5 min): 18 p/min/ft,
- Cross-flow areas: 17 p/min/ft (sum of both flows), and
- Stairways (up direction): 15 p/min/ft in the ascending direction.

SHARED-USE PATHS

Shared-use pedestrian–bicycle paths typically are open to users of nonmotorized modes such as bicyclists, skateboarders, and wheelchair users. Shared-use paths are often constructed to serve areas without city streets and to provide recreational opportunities for the public. These paths are also common on university campuses, where motor vehicle traffic and parking are often restricted. In the United States, there are few paths exclusively for pedestrians—most off-street paths are for shared use.

Bicycles—because of their markedly higher speeds—have a negative effect on pedestrian capacity and LOS on shared-use paths. However, it is difficult to establish a bicycle–pedestrian equivalent because the relationship between the two differs depending on their respective flows and directional splits, among other factors. This section covers pedestrian LOS on shared-use paths. Bicyclists have a different perspective, as discussed in the following section.

Step 1: Gather Input Data

The following input data are required for the analysis:

- Hourly pedestrian and bicycle demands by direction, and
- Average pedestrian and bicycle speeds.

LOS is based on the overtaking of pedestrians by bicyclists. Pedestrian-to-pedestrian interaction is typically negligible.

Step 2: Calculate Number of Bicycle Passing and Meeting Events

LOS for shared-use paths is based on hindrance. Research (*14*) has established LOS thresholds for pedestrians based on the frequency of passing (in the same direction) and of meeting (in the opposite direction) other users. Because pedestrians seldom overtake other pedestrians, pedestrian LOS on a shared-use path depends on the frequency with which the average pedestrian is met and overtaken by bicyclists (*14*). However, the analyst should observe pedestrian behavior in the field before assuming that pedestrian-to-pedestrian interaction is negligible. The shared-use-path methodology does not account for events with users other than bicyclists (e.g., inline skaters).

The average number of passing and meeting events per hour is calculated by Equation 23-5 and Equation 23-6. These equations do not account for the range of bicycle speeds encountered in practice; however, because of the limited degree of overlap between the speed distributions of bicyclists and pedestrians, the resulting difference is practically insignificant.

For one-way paths, there are no meeting events, so only F_p, the number of passing events, needs to be calculated. Paths 15 ft or more in width may effectively operate as two adjacent one-way facilities, in which case F_m may be set to zero.

Equation 23-5

$$F_p = \frac{Q_{sb}}{PHF}\left(1 - \frac{S_p}{S_b}\right)$$

Equation 23-6

$$F_m = \frac{Q_{ob}}{PHF}\left(1 + \frac{S_p}{S_b}\right)$$

where

F_p = number of passing events (events/h),

F_m = number of meeting events (events/h),

Q_{sb} = bicycle demand in same direction (bicycles/h),

Q_{ob} = bicycle demand in opposing direction (bicycles/h),

PHF = peak hour factor,

S_p = mean pedestrian speed on path (mi/h), and

S_b = mean bicycle speed on path (mi/h).

Meeting events create less hindrance than overtaking events.

Meeting events allow direct visual contact, so opposing-direction bicycles tend to cause less hindrance to pedestrians. To account for the reduced hindrance, a factor of 0.5 is applied to the meeting events on the basis of theoretical considerations (*14*). Where sufficient data are available on the relative effects of meetings and passings on hindrance, this factor can be calibrated to local conditions. Because the number of events calculated in the previous step was based on hourly demand, a peak hour factor must be applied to convert them to the equivalent demand based on peak 15-min conditions. The total number of events is

$$F = (F_p + 0.5F_m)$$

Equation 23-7

where F is the total events on the path in events per hour and the other variables are as defined previously.

Step 3: Determine LOS

Exhibit 23-4 is used to determine shared-use-path pedestrian LOS based on the total events per hour calculated in Step 2. Unlike the case for exclusive pedestrian facilities, the LOS E–F threshold does not reflect the capacity of a shared-use path but rather a point at which the number of bicycle meeting and passing events results in a severely diminished experience for the pedestrians sharing the path.

OFF-STREET BICYCLE FACILITIES

The uninterrupted-flow bicycle facility analysis is based on several factors that affect user perception.

On shared-use paths, the presence of other bicyclists and other path users can be detrimental to bicyclists by increasing bicycle delay, decreasing bicycle capacity, and reducing bicyclists' freedom of movement. Research (*7*) correlating user perceptions of comfort and enjoyment of path facilities with an objective measure of path and user characteristics serves as the basis for the LOS thresholds and methodology described in this section. The following key criteria are considered through this methodology:

- The ability of a bicyclist to maintain an optimum speed,
- The number of times that bicyclists meet or pass other path users, and
- The bicyclist's freedom to maneuver.

The results of a perception survey were used to fit a linear regression model in which the survey results served as the dependent variable. The methodology incorporates the effects of five path modes that may affect bicycle LOS: other bicyclists, pedestrians, runners, inline skaters, and child bicyclists. Five variables—meetings per minute, active passings per minute, path width, presence of a centerline, and delayed passings—are used in the model. In the special case of an exclusive off-street bicycle facility, the volume for all nonbicycle modes is assumed to be zero, and the number of passings and meetings is determined solely by the volume of bicycles.

On exclusive off-street bicycle facilities the number of passings and meetings is determined solely by the bicycle volume.

The following sections describe the steps to be taken in calculating bicycle LOS for an off-street facility.

Step 1: Gather Input Data

The methodology addresses five types of path users, or *mode groups*: bicyclists, pedestrians, runners, inline skaters, and child bicyclists. The following input data are required for each mode group:

- Hourly demand by direction in modal users per hour,
- Average mode group speed in miles per hour, and
- Proportion of all path users represented by a particular mode group (i.e., mode split).

In addition, the following data are required for the facility:

- Path width in feet, and
- Presence of a centerline stripe (yes or no).

With the hourly directional demand for the path and the path mode split, the hourly directional flow rate on the path is calculated for each of the five modes:

Equation 23-8

$$q_i = \frac{Q_T \times p_i}{PHF}$$

where

q_i = hourly directional path flow rate for user group i (modal users/h),

Q_T = total hourly directional path demand (modal users/h),

p_i = path mode split for user group i, and

PHF = peak hour factor.

Step 2: Calculate Active Passings per Minute

Active passings are defined as the number of other path users traveling in the same direction as an average bicyclist (i.e., a bicyclist traveling at the average speed of all bicycles), who are passed by that bicyclist. The average bicyclist is assumed to move at a constant speed U. The value of U should be set to the average speed of bicyclists on the facility in question; where local data are not available, the default average bicyclist speed of 12.8 mi/h may be used. The methodology for determining active passings incorporates separately the effects of each of the five mode groups described in Step 1. The speeds of path users of each mode group are assumed to be normally distributed with a mean μ_i and standard deviation σ_i^2, where i represents mode.

The average bicyclist passes only those users who (*a*) are present on the path segment when the average bicyclist enters and (*b*) exit the segment after the average bicyclist does. Thus, for a given modal user in the path when the average bicyclist enters, the probability of being passed is expressed by

Equation 23-9

$$P(v_i) = P[v_i < U(1 - \frac{x}{L})]$$

where

$P(v_i)$ = probability of passing user of mode i,

U = speed of average bicyclist (mi/h),

v_i = speed of path user of mode i (mi/h),

L = length of path segment (mi), and

x = distance from average bicyclist to user (mi).

Exhibit 23-12 provides a schematic of active passing events.

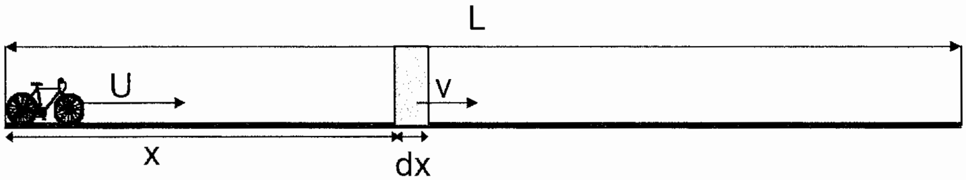


Exhibit 23-12
Schematic of Active Passing Events

Since v_i is distributed normally, the probability in Equation 23-9 can be calculated from the integral under the standard normal curve. By dividing the full length of the path L into n small discrete pieces each of length dx, the average probability of passing within each piece j can be estimated as the average of the probabilities at the start and end of each piece:

$$P(v_i) = 0.5[F(x - dx) + F(x)]$$

Equation 23-10

where $F(x)$ is the cumulative probability of normal distribution, and the other variables are as defined previously.

The expected number of times that the average bicyclist passes users of mode i over the entire path segment is then determined by multiplying $P(v_i)$ by the density of users of mode i and summing over all portions of the segment. The number of passings per minute is then obtained by dividing the result by the number of minutes required for the bicyclist to traverse the path segment:

$$A_i = \sum_{j=1}^{n} P(v_i) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

Equation 23-11

where

A_i = expected passings per minute of mode i by average bicyclist,

q_i = directional hourly flow rate of mode i (modal users/h),

μ_i = average speed of mode i (mi/h),

t = path segment travel time for average bicyclist (min), and

dx_j = length of discrete segment j (mi).

The other variables are as previously defined.

Research (*7*) found that setting dx equal to 0.01 mi is appropriate for the purposes of the calculations shown in Equation 23-11 and below.

Equation 23-11 provides expected passings by the average bicyclist for mode i. To determine total active passings of all modes, Equation 23-11 must be repeated for each individual mode and then summed:

$$A_T = \sum_i A_i$$

Equation 23-12

where A_T is the expected active passings per minute by the average bicyclist during the peak 15 min, and the other variables are as defined previously.

Step 3: Calculate Meetings per Minute

Meetings are defined as the number of path users traveling in the opposing direction to the average bicyclist that the average bicyclist passes on the path segment. All users present on the path when the average bicyclist enters will be passed by the average bicyclist, assuming that no user enters or exits the path at an intermediate point:

Equation 23-13

$$M_1 = \frac{U}{60}\sum_i \frac{q_i}{\mu_i}$$

where M_1 is the meetings per minute of users already on the path segment and U is the speed of the average bicyclist in miles per hour. The other variables are as previously defined.

In addition to users already on the path segment, a number of users who have yet to enter the segment will meet the average bicyclist within the segment. The probability of this occurrence is

Equation 23-14

$$P(v_{Oi}) = P(v_i > X\frac{U}{L})$$

where

$P(v_{Oi})$ = probability of meeting opposing user of mode i,

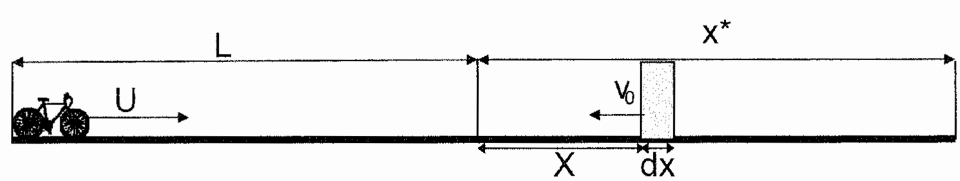
v_i = speed of path user of mode i (mi/h),

X = distance of user beyond end of path segment (mi), and

U = speed of average bicyclist (mi/h).

Since v_{Oi} is distributed normally, the probability in Equation 23-14 can be readily calculated from the area under the standard normal curve. The length of path beyond the analysis segment that may supply users who will be met by the average bicyclist is defined as x^*. By dividing x^* into n small discrete pieces, each of length dx, the average probability of meeting a modal user from each piece can be estimated by Equation 23-10. Although some meetings will occur with very fast path users located greater than L distance beyond the end of the segment when the average bicyclist enters, setting x^* equal to L is sufficient to guarantee that at least 99% of meetings will be captured (7). Exhibit 23-13 provides a schematic of meeting events.

Exhibit 23-13
Schematic of Meeting Events



Similar to the process for calculating number of active passings (Equation 23-11), the estimation of number of meetings with users from a particular mode group not on the path segment when the average bicyclist enters is

Equation 23-15

$$M_{2i} = \sum_{j=1}^{n} P(v_{Oi}) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

where M_{2i} is the expected meetings per minute of users of mode i located beyond the end of the path segment at the time the average bicycle enters the segment, and the other variables are as previously defined.

Finally, the total number of expected meetings per minute during the peak 15 min M_T is determined by adding M_1 to the sum of M_{2i} across all mode groups:

Equation 23-16

$$M_T = (M_1 + \sum_i M_{2i})$$

All variables are as previously defined.

In the special case of a one-way path, there are no opposing users to meet; therefore, M_T is zero.

Step 4: Determine Number of Lanes

The effective number of lanes on a shared-use path affects the number of delayed passings: as the number of lanes increases, delayed passings decrease. Even paths without painted lane markings will operate with a de facto number of lanes. The relationship between path width and the number of effective operational lanes is shown in Exhibit 23-14.

Exhibit 23-14
Effective Lanes by Path Width

Path Width (ft)	Lanes
8.0–10.5	2
11.0–14.5	3
15.0–20.0	4

Source: Hummer et al. (7).

Step 5: Calculate Probability of Delayed Passing

Delayed passing maneuvers occur when there is a path user ahead of the overtaking average bicyclist in the subject direction and another path user in the opposing direction, such that the average bicyclist cannot immediately make the passing maneuver. The probability of a delayed passing depends on the passing distance required, which in turn depends on both the overtaking mode and the mode of the user being passed. The passing distances that bicyclists require to pass other user modes are shown in Exhibit 23-15.

Exhibit 23-15
Required Bicycle Passing Distance

Overtaking Mode	Mode Passed	Required Passing Distance (ft)
Bicycle	Bicyclist	100
Bicycle	Pedestrian	60
Bicycle	Inline skater	100
Bicycle	Runner	70
Bicycle	Child bicyclist	70

Source: Hummer et al. (7).

With the values in Exhibit 23-15, the probability that a given section will be vacant of a given mode for at least the required passing distance p_i can be estimated by using a Poisson distribution. The probability of observing at least one modal user in the passing section is the complement of the probability of observing a vacant section. The probability P_{ni} of observing a blocked passing section for mode i is

Equation 23-17

$$P_{ni} = 1 - e^{-p_i k_i}$$

where

P_{ni} = probability of passing section's being blocked by mode i,

p_i = distance required to pass mode i (mi), and

k_i = density of users of mode i (users/mi).

Equation 23-17 is applicable to both the subject and opposing directions.

Two-Lane Paths

On a two-lane path, delayed passing occurs when, within the distance required to complete a pass p, the average bicyclist encounters one of the following: traffic in both directions, each blocking a single lane, or no traffic in the subject direction in conjunction with traffic in the opposing direction that is being overtaken by an opposing bicyclist. Note that these situations are mutually exclusive. The delayed passing probabilities in the subject and opposing directions are

Equation 23-18

$$P_{ds} = P_{no}P_{ns} + P_{no}(1-P_{ns})(1-P_{do})$$

Equation 23-19

$$P_{do} = P_{no}P_{ns} + P_{ns}(1-P_{no})(1-P_{ds})$$

where

P_{ds} = probability of delayed passing in subject direction,

P_{do} = probability of delayed passing in opposing direction,

P_{no} = probability of blocked lane in opposing direction, and

P_{ns} = probability of blocked lane in subject direction.

Solving Equation 23-18 and Equation 23-19 for P_{ds} results in

Equation 23-20

$$P_{ds} = \frac{P_{no}P_{ns} + P_{no}(1-P_{ns})^2}{1-P_{no}P_{ns}(1-P_{no})(1-P_{ns})}$$

Since P_{no} and P_{ns} are calculated from Equation 23-17, Equation 23-20 can be readily solved for P_{ds}.

Three-Lane Paths

The operations of three-lane paths are more complicated than those of two-lane paths, since there is a greater variety of possible scenarios that may occur. The methodology includes several limiting assumptions regarding user behavior:

- Bicyclists in the subject direction use only the two rightmost lanes,
- Bicyclists in the opposing direction use only the two leftmost lanes,
- Passing maneuvers occur only in the middle lane and never in the left lane, and
- Groups of users may sometimes block the two lanes allocated to that direction but cannot block all three lanes.

As a result, a delayed passing occurs in two cases: (*a*) traffic in the subject direction is blocking the rightmost lane in conjunction with opposing traffic occupying the other two lanes, or (*b*) side-by-side users are blocking the two rightmost lanes in the subject direction. The probabilities of the occurrence of a delayed passing in the subject and opposing directions are given by

$$P_{ds} = P_{ns}[P_{bo} + P_{no}(1 - P_{do})] + P_{bs}$$ Equation 23-21

$$P_{do} = P_{no}[P_{bs} + P_{ns}(1 - P_{ds})] + P_{bo}$$ Equation 23-22

where P_{bo} is the probability of two blocked lanes in the opposing direction, P_{bs} is the probability of two blocked lanes in the subject direction, and all other variables are as previously defined.

Equation 23-21 and Equation 23-22 are simultaneous equations with two unknowns, P_{ds} and P_{do}. Defining D as $P_{ds} - P_{do}$ gives the following equation:

$$D = [(P_{bs} - P_{bo}) + (P_{ns}P_{bo} - P_{no}P_{bs})]/(1 - P_{ns}P_{no})$$ Equation 23-23

Substituting Equation 23-23 into Equation 23-21 results in

$$P_{ds} = [P_{ns}(P_{bo} + P_{no}(1 + D)) + P_{bs}]/(1 + P_{ns}P_{no})$$ Equation 23-24

This model requires determining four probability parameters: specifically, P_n and P_b in each direction. Calculating these parameters requires estimating the fraction of all events in which both lanes are blocked. These parameters were established through research (7) in which video data of more than 4,000 path users on U.S. shared-use paths were observed. Exhibit 23-16 shows the blocking frequencies by mode.

Exhibit 23-16
Frequency of Blocking of Two Lanes

Mode	Frequency of Blocking (%)
Bicycle	5
Pedestrian	36
Inline skater	8
Runner	12
Child bicyclist	1

Source: Hummer et al. (7).

Therefore, P_{boi} and P_{bsi}, the probabilities that a user of mode i will block two lanes in the opposing and subject directions, respectively, are found by multiplying the frequency of blocking two lanes by a particular user of mode i (Exhibit 23-16) by the probability that a user of mode i will be encountered, which was given by Equation 23-17. This process results in

$$P_{bsi} = F_i \times P_{nsi}$$ Equation 23-25

$$P_{boi} = F_i \times P_{noi}$$ Equation 23-26

where F_i is the frequency with which mode i will block two lanes, from Exhibit 23-16, and all other variables are as previously defined. The probability that a user of any mode will block two lanes is thus given by

$$P_{bs} = \sum_i P_{bsi}$$ Equation 23-27

$$P_{bo} = \sum_i P_{boi}$$ Equation 23-28

The probabilities that only a single lane will be blocked by a user of a given mode i, P_{qsi} and P_{qoi}, are thus derived from the probability that at least one lane will be blocked (from Equation 23-17) minus the probability that two lanes will be blocked (from Equation 23-25 and Equation 23-26). These probabilities are

Equation 23-29

$$P_{nsi} = 1 - e^{-p_i k_{si}} - P_{bsi}$$

Equation 23-30

$$P_{noi} = 1 - e^{-p_i k_{oi}} - P_{boi}$$

where k_{si} and k_{oi} are the densities of users of mode i in users per mile in the subject and opposing directions, respectively, and all other variables are as previously defined.

The probabilities that a user of any mode will block a single lane are thus given by

Equation 23-31

$$P_{ns} = \sum_i P_{nsi}$$

Equation 23-32

$$P_{no} = \sum_i P_{noi}$$

The values of P_{bs} and P_{bo} from Equation 23-27 and Equation 23-28, and the values of P_{ns} and P_{no} from Equation 23-31 and Equation 23-32 can now be substituted into Equation 23-23 and Equation 23-24 to determine the probability of delayed passing, P_{ds}. This delayed passing factor was calibrated by using peak hour volumes, rather than peak 15-min volumes. Therefore, a peak hour factor is applied to convert A_T from peak 15-min flow rate conditions back to hourly conditions.

Four-Lane Paths

On four-lane paths, the methodology assumes that the path operates similarly to a divided four-lane highway, such that the probability of delayed passing is independent of opposing users, since no passing occurs in the leftmost lanes. Thus, the probability of delayed passing P_{ds} is equivalent to the probability that both subject lanes will be blocked P_{bs}, which can be found by using Equation 23-25 and Equation 23-27.

Step 6: Calculate Delayed Passings per Minute

The probability of delayed passing P_{ds}, described earlier, applies only to a single pair of modal path users (e.g., a bicyclist passing a pedestrian and opposed by a runner). The total probability of delayed passing P_{Tds} must be calculated from all modal pairs. Since there are five modes, there are five times five (25) total modal pairs that require calculation. The total probability of delayed passing is found by using

Equation 23-33

$$P_{Tds} = 1 - \prod_m (1 - P_{mds})$$

where P_{Tds} is the total probability of delayed passing and P_{mds} is the probability of delayed passing for mode m. The operator Π in Equation 23-33 indicates the product of a series of variables.

Finally, delayed passings per minute are simply the active passings per minute A_T multiplied by the total probability of delayed passing P_{Tds}:

Equation 23-34

$$\text{Delayed Passings per Minute} = A_T \times P_{Tds} \times PHF$$

This delayed passing factor was calibrated by using peak hour volumes rather than peak 15-min volumes. Therefore, a peak hour factor is applied to convert A_T from peak 15-min flow rate conditions back to hourly conditions.

Step 7: Determine LOS

The bicycle LOS score (Equation 23-35) uses inputs from Steps 2, 3, and 6 plus facility data gathered in Step 1. The equation was developed from a regression model of user responses to video clips depicting a variety of off-street bicycle facilities (7). The LOS C–D threshold represents the midpoint of the response scale used in the survey.

$$\text{Bicycle LOS Score} = 5.446 - 0.00809(E) - 15.86(RW) - 0.287(CL) - 0.5(DP)$$

Equation 23-35

where

E = weighted events per minute = meetings per minute + 10 × (active passings per minute);

RW = reciprocal of path width = 1/path width (ft);

CL = 1 if trail has centerline, 0 if no centerline; and

DP = min [delayed passings per minute, 1.5].

With the exception of the special cases discussed in Step 8 below, the bicyclist perception index is used directly with Exhibit 23-5 to determine bicyclist LOS on off-street facilities. As was the case with shared pedestrian facilities, the LOS E–F threshold does not reflect the capacity of an off-street bicycle facility, but rather a point at which the number of meeting and passing events results in a severely diminished experience for bicyclists using the path.

Step 8: Adjust LOS for Low-Volume Paths

For narrow paths (i.e., 8 ft in width), it is not possible to achieve LOS A or B by using Equation 23-35. Since paths with very low volumes would be expected to result in a high perceived quality of service, the following adjustments are made to the LOS results:

- All paths with five or fewer weighted events per minute are assigned LOS A.
- All paths with >5 to 10 weighted events per minute are assigned LOS B, unless Equation 23-35 would result in LOS A.

3. APPLICATIONS

This chapter's methodologies evaluate the LOS of exclusive pedestrian and bicycle facilities. The analyst must address two fundamental issues. First, the primary outputs must be identified. These may include LOS, effective width W_E, or achievable path flow rate Q_T. Second, any necessary default or estimated values must be identified. There are three basic sources of input data:

1. Default values provided in the HCM;
2. Estimates or locally derived default values developed by the user; and
3. Values derived from field measurements and observation.

When it is possible to obtain them, field measurements are preferable to default values.

DEFAULT VALUES

For pedestrians on off-street paths, a default average speed of 3.4 mi/h for pedestrians and 12.8 mi/h for bicycles can be applied in the absence of local data (7). Default values for off-street bicycle facilities are summarized in Exhibit 23-17:

Exhibit 23-17
Default Values for Exclusive Off-Street Bicycle Facilities

Variable	User Group	Default Value
Mode split	Bicycle	55%
	Pedestrian	20%
	Runner	10%
	Inline skater	10%
	Child bicyclist	5%
Mean speed by mode	Bicycle	12.8 mi/h
	Pedestrian	3.4 mi/h
	Runner	6.5 mi/h
	Inline skater	10.1 mi/h
	Child bicyclist	7.9 mi/h
Standard deviation of speed by mode	Bicycle	3.4 mi/h
	Pedestrian	0.6 mi/h
	Runner	1.2 mi/h
	Inline skater	2.7 mi/h
	Child bicyclist	1.9 mi/h
Proportion of users blocking two lanes by mode	Bicycle	5%
	Pedestrian	36%
	Runner	12%
	Inline skater	8%
	Child bicyclist	1%
Peak hour factor	N/A	0.85

Source: Hummer et al. (*7*).

ANALYSIS BOUNDARIES

As stated in this chapter's introduction, exclusive pedestrian and bicycle facilities are analyzed at the segment level, with segment endpoints being defined by street crossing locations, changes in path width, intersections with other paths that create cross flows or change path demand, and transition points to other types of facilities (e.g., from path to ramp). In most cases, the minimum segment length will be around 0.25 mi, and the maximum segment length will be 2 to 3 mi (7). Certain kinds of facilities, such as stairways, cross-flow areas, and pedestrian plazas, will have shorter segment lengths.

TYPES OF ANALYSIS

Operational Analysis

A common application of operational analysis is to compute the LOS of a facility under existing or future demand. The effective width of the facility is an input to the calculation and LOS is an output.

Design Analysis

Designing for an effective width.

Design applications require that an LOS goal be established, with the primary output being the facility design characteristics required or the maximum user volumes allowable for the LOS goal. For instance, a design analysis for a pedestrian walkway may estimate the minimum effective width W_E needed to achieve a design LOS value. In this case, the maximum pedestrian unit flow rate for the desired service level would be determined from Exhibit 23-1 or Exhibit 23-2. The effective width would be computed by solving the pedestrian unit flow-rate equation backward. To avoid pedestrian spillover, it is desirable to design a walkway to achieve LOS C or better (i.e., a maximum of 10 p/min/ft). Stairways are desirably designed to achieve LOS C or D.

Determining service volumes.

Similarly, the achievable path flow rate Q_T can be solved as the primary output. For exclusive bicycle facilities, the minimum LOS perception score for the design LOS would be determined from Exhibit 23-5. By holding all but one path user group's demand constant and solving the events equation backward, the service volume for the user group of interest can be computed.

Planning and Preliminary Engineering Analyses

Planning and preliminary engineering analyses use estimates, HCM default values, or local default values as inputs and determine LOS, bicycle flow rate, effective width, or all three, as outputs. The difference between a planning analysis and an operational or design analysis is that most or all of the input values in planning come from estimates or default values, whereas operational and design analyses tend to use field measurements or known values for most or all of the input variables.

SPECIAL CASES

Pedestrian Plazas

Pedestrian plazas are large, paved areas that serve multiple functions, including pedestrian circulation, special events, and seating. The circulation function is of interest here, although the design of a plaza must consider how all of the functions interact. For example, queues from areas designated for food vendors may intrude into a pedestrian circulation route, reducing the route's effective width, or two circulation routes may intersect each other, creating a cross-flow area. In addition, research has shown that the circulation and amenity functions of a plaza sometimes conflict, since people tend to linger longer in plazas that do not act as thoroughfares (*9*).

The exclusive pedestrian walkway methodology can be used to analyze pedestrian circulation routes through pedestrian plazas. The methodology does not address the need or desire to have space for amenities within a pedestrian plaza. The effective width of such a route is not as easily identified as that of a walkway, because the edges of the circulation area are often undefined. However, pedestrians will tend to take the shortest available route across the plaza, as illustrated in Exhibit 23-18.

Exhibit 23-18
Pedestrian Circulation Space in a Pedestrian Plaza

The effective width of a circulation route is influenced by the widths of the entrance and exit points to the plaza and by the presence of obstacles (e.g., walls, poles, signs, benches). Effective width may also be influenced by whether a change in texture is used to mark the transition between circulation and amenity space. Between 30% and 60% of pedestrians will use plaza space that is flush with a sidewalk, with the higher percentages applying to wider plazas and those that help cut a corner and the lower percentages applying to narrower plazas and those with obstacles (*9*).

For design applications, peak pedestrian demands through the plaza would need to be estimated. Given this information and a design LOS, a minimum effective width could be determined for each circulation route. Multiplying the width of the route by the length of the route and summing for all routes results in the space required for pedestrian circulation. Space requirements for sitting areas and other plaza functions are added to the circulation space to determine the total plaza space required.

For operational applications, an average effective width can be determined through field observation of the space occupied by pedestrians on a circulation route during peak times. Dividing an average per minute pedestrian volume by the effective width gives the pedestrian flow rate for the circulation route, from which LOS can be determined.

Pedestrian Zones

Pedestrian zones are streets dedicated to exclusive pedestrian use on a full- or part-time basis. These zones can be analyzed from an operational standpoint by using the exclusive pedestrian walkway methodology, as long as the kinds of obstructions listed in Exhibit 23-11, such as sidewalk café tables, are taken into account. Other performance measures may be considered that assess the street's attractiveness to pedestrians, since a successful pedestrian zone is expected to be relatively crowded (i.e., to have a lower LOS). Although an uncrowded zone would have a high LOS, it could be perceived by pedestrians as being a potential personal security risk, because of the lack of other users.

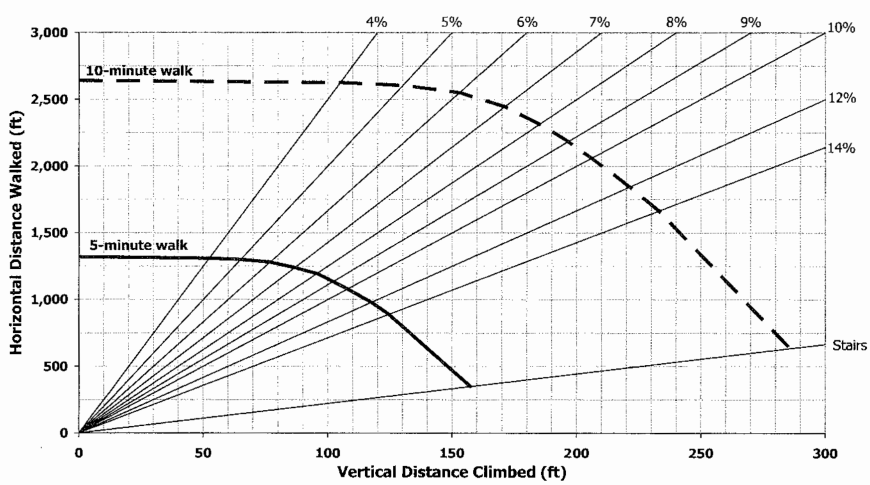
The HCM methodology is not suitable for pedestrian zones during times when delivery vehicles are allowed to use the street. The HCM methodology is also not applicable to the analysis of a low-speed street (e.g., a Dutch-style *woonerf*) shared by pedestrians, bicycles, and automobiles.

Walkways with Grades over 5%

Research (*9–11*) has shown no appreciable impact on pedestrian speed for grades up to 5%. As shown in Exhibit 23-19, above a 5% grade, walking speeds drop as grade increases, with travel on a 12% grade being about 30% slower than travel on a level surface. Grade may not have an appreciable impact on capacity, however, since the reduction in pedestrian speed is offset by closer pedestrian spacing (*9*). The stairway LOS table (Exhibit 23-3) would provide a conservative estimate of pedestrian LOS on steeper walkways.

Consult the latest version of the ADA Accessibility Guidelines for guidance on the maximum slope allowed on an accessible route.

Exhibit 23-19
Effect of Vertical Climb on Horizontal Distance Walked



Source: Municipal Planning Association (*11*).

Paths Segregating Pedestrians and Bicyclists

Some paths are signed or striped, or both, to segregate bicyclists from pedestrians. Where field observation on the path (or similar paths in the same region) indicates that path users generally comply with the regulations, up to all of the bicycle–pedestrian passing events could be converted to meeting events in proportion to the path users' compliance rate, resulting in an improved LOS. Where sufficient physical segregation of bicyclists and pedestrians occurs, it may be appropriate to treat the path as two separate facilities.

USE OF ALTERNATIVE TOOLS

To date, there is no widely used computer simulation software in the United States that is capable of describing user interactions on shared-use paths in a realistic manner. Microsimulation has been used to model pedestrian interactions on off-street pedestrian facilities. In many cases, these models were developed to model pedestrian movements within airports or transit facilities.

4. EXAMPLE PROBLEMS

Exhibit 23-20
List of Example Problems

Example Problem	Description	Application
1	Pedestrian LOS on shared-use and exclusive paths	Operational analysis
2	Bicycle LOS on a shared-use path	Planning analysis

EXAMPLE PROBLEM 1: PEDESTRIAN LOS ON SHARED-USE AND EXCLUSIVE PATHS

The Facts

The following information was collected in the field for this path:

- Q_{sb} = bicycle volume in same direction = 100 bicycles/h;
- Q_{ob} = bicycle volume in opposing direction = 100 bicycles/h;
- v_{15} = peak 15-min pedestrian volume = 100 pedestrians;
- PHF = peak hour factor = 0.83;
- S_p = average pedestrian speed = 4.0 ft/s (2.7 mi/h);
- S_b = average bicycle speed = 16.0 ft/s (10.9 mi/h); and
- No pedestrian platooning was observed.

Step 1: Gather Input Data

The shared-use-path pedestrian LOS methodology requires pedestrian and bicycle speeds and bicycle demand, all of which are available from the field measurements just given.

Step 2: Calculate Number of Bicycle Passing and Meeting Events

The number of passing events F_p is determined from Equation 23-5:

$$F_p = \frac{Q_{sb}}{PHF}\left(1 - \frac{S_p}{S_b}\right)$$

$$F_p = \frac{100 \text{ bicycles/h}}{0.83}\left(1 - \frac{4.0 \text{ ft/s}}{16.0 \text{ ft/s}}\right)$$

$$F_p = 90 \text{ events/h}$$

The number of meeting events F_m is determined from Equation 23-6:

$$F_m = \frac{Q_{ob}}{PHF}\left(1 + \frac{S_p}{S_b}\right)$$

$$F_m = \frac{100 \text{ bicycles/h}}{0.83}\left(1 + \frac{4.0 \text{ ft/s}}{16.0 \text{ ft/s}}\right)$$

$$F_m = 150 \text{ events/h}$$

The total number of events is calculated from Equation 23-7:

$$F = (F_p + 0.5F_m)$$

$$F = (90\text{ events/h} + (0.5)(150\text{ events/h}))$$

$$F = 165\text{ events/h}$$

Step 3: Determine Shared-Use-Path Pedestrian LOS

The shared-use-path LOS is determined from Exhibit 23-4. The value of F, 165 events/h, falls into the LOS E range. Since this value is rather low, what would happen if a parallel, 5-ft-wide, pedestrian-only path were provided?

Step 4: Compare Exclusive-Path Pedestrian LOS

Step 4.1: Determine Effective Walkway Width

Assuming that no obstacles exist on or immediately adjacent to the path, the effective width would be the same as the actual width, or 5 ft. Common amenities located along pedestrian walkways include trash cans and benches. From Exhibit 23-11, these should be located at least 3.0 ft and 5.0 ft, respectively, from the edge of the path to avoid affecting the effective width.

Step 4.2: Calculate Pedestrian Flow Rate

Since a peak-15-min pedestrian volume was measured in the field, it is not necessary to use Equation 23-2 to determine v_{15}. The unit flow rate for the walkway v_p is determined from Equation 23-3 as follows:

$$v_p = \frac{v_{15}}{15 \times W_E}$$

$$v_p = \frac{100\text{ p}}{15 \times 5\text{ ft}}$$

$$v_p = 1.3\text{ p/ft/min}$$

Step 4.3: Calculate Average Pedestrian Space

Average pedestrian space is determined by rearranging Equation 23-4:

$$A_p = S_p / v_p$$

$$A_p = (4.0\text{ ft/s})(60\text{ s/min}) / (1.3\text{ p/ft/min})$$

$$A_p = 185\text{ ft}^2\text{/p}$$

Step 4.4: Determine LOS

Since no pedestrian platooning was observed, Exhibit 23-1 should be used to determine the LOS. A value of 185 ft^2/min corresponds to LOS A.

Discussion

The existing mixed-use path operates at LOS E for pedestrians. Pedestrian LOS would increase to LOS A if a parallel, 5-ft-wide pedestrian path were provided.

EXAMPLE PROBLEM 2: BICYCLE LOS ON A SHARED-USE PATH

The Facts

A new shared-use path is being planned. On the basis of data from a similar facility in the region, it is estimated that the path will have a peak-hour volume of 340 users, a peak hour factor of 0.90, and a 50/50 directional split. The path will be 10 ft wide, with no obstacles, and will not have a centerline. The segment analyzed here is 3 mi long.

Step 1: Gather Input Data

Facility and overall demand data are available but not the mode split of users or the average mode group speed. Those values will need to be defaulted by using Exhibit 23-17. On the basis of the default mode split and the estimated directional split, the directional hourly volume by mode is as follows:

- Directional bicycle flow rate = 340 users/h × 0.5 × 0.55/0.90 = 104 bicycles/h;
- Directional pedestrian flow rate = 340 × 0.5 × 0.20/0.90 = 38 p/h;
- Directional runner flow rate = 340 × 0.5 × 0.10/0.90 = 19 runners/h;
- Directional inline skater flow rate = 340 × 0.5 × 0.10/0.90 = 19 skaters/h; and
- Directional child bicyclist volume = 340 × 0.5 × 0.05/0.90 = 9 child bicyclists/h.

From Exhibit 23-17, average mode group speeds and standard deviations are as follows:

- Bicycle: average speed = 12.8 mi/h, standard deviation = 3.4 mi/h;
- Pedestrian: average speed = 3.4 mi/h, standard deviation = 0.6 mi/h;
- Runner: average speed = 6.5 mi/h, standard deviation = 1.2 mi/h;
- Inline skater: average speed = 10.1 mi/h, standard deviation = 2.7 mi/h; and
- Child bicyclist: average speed = 7.9 mi/h, standard deviation = 1.9 mi/h.

Step 2: Calculate Active Passings per Minute

Active passings per minute must be calculated separately for each mode, by using Equation 23-9 through Equation 23-11. For the number of bicycles passed per minute, the path is considered as broken into n slices, each of which has a length dx of 0.01 mi, and a total path segment length L of 3 mi. Then, for the first slice, from Equation 23-9:

$$P(v_i) = P[v_i < U(1-\frac{x}{L})]$$

$$F(x) = P[v_i < 12.8(1-\frac{0.01}{3})] = P[v_i < 12.76] = 0.4949$$

$$F(x-dx) = P[v_i < 12.8(1-\frac{0}{3})] = P[v_i < 12.80] = 0.5000$$

Applying Equation 23-10 and Equation 23-11 then gives the following probability of passing for the first slice:

$$P(v_i) = 0.5[F(x-dx) + F(x)]$$

$$P(v_i) = 0.5[0.5000 + 0.4949] = 0.4975$$

$$A_i = \sum_{j=1}^{n} P(v_i) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

$$A_{i1} = 0.4975 \times \frac{104}{12.8} \times \frac{1}{14}(0.01) = 0.0029$$

Repeating this procedure for all slices from 1 to n and summing the results yields

Passings of bicycles per minute = $0.0029 + A_2 + \ldots + A_n = 0.19$

With the same methodology for each mode, the following active passings per minute are found for the other modes:

- Pedestrians, 1.74;
- Runners, 0.30;
- Inline skaters, 0.09; and
- Child bicyclists, 0.10.

Total active passings are then determined by using Equation 23-12:

$$A_T = \sum_i A_i$$

Total passings per minute = (0.19 + 1.74 + 0.30 + 0.09 + 0.10) = 2.42

Step 3: Calculate Meetings per Minute

Meetings per minute of users already on the path segment M_1 are calculated for each mode with Equation 23-13:

$$M_1 = \frac{U}{60} \sum_i \frac{q_i}{\mu_i}$$

$$M_1 = (12.8/60) \times [(104/12.8) + (38/3.4) + (19/6.6) + (19/10.1) + (9/7.9)] = 5.36$$

Meetings per minute of users not yet on the path segment must be calculated separately for each mode. For the number of bicycles passed per minute, the section of path beyond the study segment is considered as broken into n slices, each of which has length $dx = 0.01$ mi, and a total segment length X equivalent to L (3 mi). Then, for the first slice, from Equation 23-14:

$$P(v_{Oi}) = P(v_i > X\frac{U}{L})$$

$$F(x) = P[v_i > 0.01 \times \frac{12.8}{3}] = P[v_i > 0.4267] = 0.9999$$

$$F(x) = P[v_i > 0 \times \frac{12.8}{3}] = P[v_i > 0] = 1.0000$$

Applying Equation 23-10 and Equation 23-15 then gives the probability of passing in the first slice:

$$P(v_i) = 0.5[F(x - dx) + F(x)]$$

$$P(v_i) = 0.5\ [0.99992 + 1.0000] = 0.99996$$

$$M_{2i} = \sum_{j=1}^{n} P(v_{Oi}) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

$$M_{21} = 0.99996 \times (104/12.8) \times (1/14) \times 0.01 = 0.0058$$

Repeating this procedure for all slices from 1 to n and summing the results yields

$$M_{2i} = \text{meetings of bicycles per minute} = 0.0058 + M_{22} + \ldots + M_{2n} = 1.54$$

Repeating the foregoing procedure for the other modes, the following meetings per minute are found for each mode:

- Pedestrians, 0.63;
- Runners, 0.31;
- Inline skaters, 0.31; and
- Child bicyclists, 0.16.

Total meetings are then determined by using Equation 23-16:

$$M_T = (M_1 + \sum_i M_{2i})$$

$$\text{Total meetings per minute} = [5.36 + 1.54 + 0.63 + 0.31 + 0.31 + 0.16] = 8.31$$

Step 4: Determine the Number of Lanes

From Exhibit 23-14, a 10-ft-wide path has two effective lanes.

Step 5: Calculate the Probability of Delayed Passing

From Step 4, it is clear that a path with a width of 10 ft will operate as two lanes. Therefore, delayed passings per minute must be calculated separately for each of the 25 modal pairs, by using Equation 23-17 and Equation 23-20. For instance, considering the probability of a delayed passing of a bicyclist as a result of an opposing bicyclist's overtaking a pedestrian gives the following:

$$P_{ni} = 1 - e^{-p_i k_i}$$

$$P_{ns} = 1 - \exp[-(100/5280) \times (1/0.90\ (94/12.8))] = 1 - 0.858 = 0.142$$

$$P_{no} = 1 - \exp[-(100/5280) \times (1/0.90\ (38/3.4))] = 1 - 0.810 = 0.190$$

Substituting into Equation 23-20 then yields P_{ds}:

$$P_{ds} = \frac{P_{no}P_{ns} + P_{no}(1-P_{ns})^2}{1-P_{no}P_{ns}(1-P_{no})(1-P_{ns})}$$

$$P_{ds} = \frac{0.190 \times 0.142 + 0.190(1-0.142)^2}{1-0.190 \times 0.142(1-0.190)(1-0.142)} = 0.1698$$

Performing the foregoing procedures for each of the 25 modal pairs and applying Equation 23-33 gives the total probability of delayed passing:

$$P_{Tds} = 1 - \prod_m 1 - P_{mds}$$

$$P_{Tds} = 1 - (1 - 0.1698)(1 - P_{2ds}) \ldots (1 - P_{mds}) = 0.8334$$

Thus, the probability of delayed passing is 83.34%.

Step 6: Determine Delayed Passings per Minute

Equation 23-34 is used to determine the total number of delayed passings per minute:

$$\text{Delayed passings per minute} = A_T \times P_{Tds} \times PHF$$

$$\text{Total delayed passings per minute} = 0.8334 \times 2.42 \times 0.90 = 1.82$$

The delayed passing factor has a maximum of 3. The total delayed passings per minute (1.82) is less than 3. Therefore the delayed passing factor is set to 1.82.

Step 7: Calculate LOS

Equation 23-35 is used to determine the bicycle LOS score for the path:

$$\text{Bicycle LOS Score} =$$
$$5.446 - 0.00809\,[8.58 + (10 \times 2.42)] - 15.86(1/10) - 0.287(0) - 0.5(1.82) = 2.69$$

Because the bicyclist perception index is between 2.5 and 3.0, the facility operates at LOS D according to Exhibit 23-5.

Results

The results indicate that the path would operate close to its functional capacity. A slightly wider path would provide three effective lanes and a better LOS.

Many of these references can be found in the Technical Reference Library in Volume 4.

5. REFERENCES

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VOLUME 3 INDEX

The index to Volume 3 lists the text citations of the terms defined in the Glossary (Volume 1, Chapter 9). Volumes 1, 2, and 3 are separately indexed. In the index listings, the first number in each hyphenated pair of numbers indicates the chapter, and the number after the hyphen indicates the page within the chapter.

A

B

C

D

E

F

G

H

T

U

V

W