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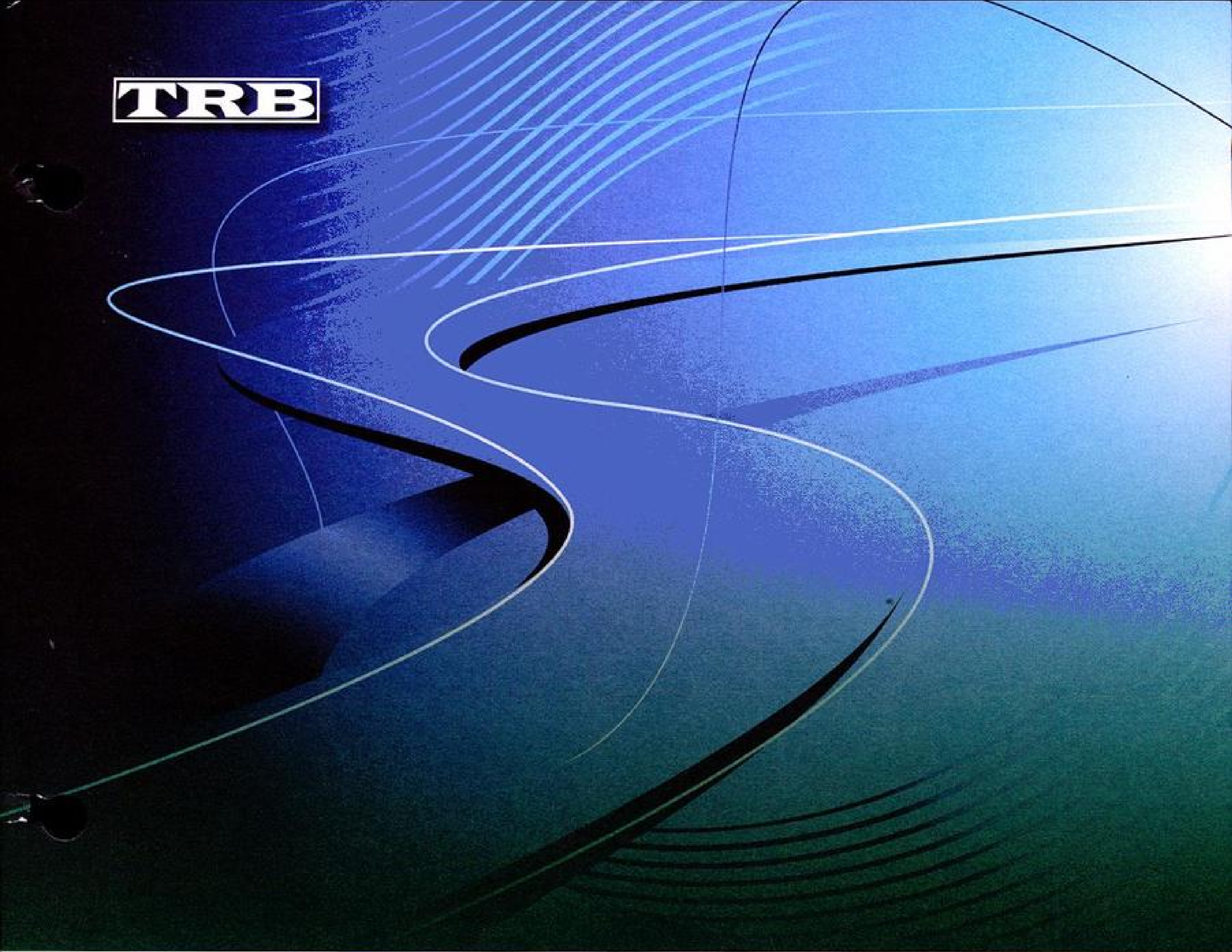

HIGHWAY CAPACITY MANUAL

6TH EDITION | A GUIDE FOR MULTIMODAL MOBILITY ANALYSIS

VOLUME 3: INTERRUPTED FLOW

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Printed in the United States of America.

ISBN 978-0-309-36997-8 [Slipcased set of three volumes]
ISBN 978-0-309-36998-5 [Volume 1]
ISBN 978-0-309-36999-2 [Volume 2]
ISBN 978-0-309-37000-4 [Volume 3]
ISBN 978-0-309-37001-1 [Volume 4, online only]

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CHAPTER 16
URBAN STREET FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

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

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 methodologies described in this chapter are intended to assist the analyst by providing a means of assessing the performance of each travel mode that takes account of the influence of other modes.

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

Each methodology in this chapter is focused on the evaluation of an urban street facility that is made up of two or more segments. A separate methodology for evaluating the performance of individual segments is described in Chapter 18, Urban Street Segments. The performance measures associated with each segment are then aggregated to the facility level with the methodology described in this chapter.

A facility's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road-user group. Performance measures cited in this chapter include motorized vehicle travel speed, motorized vehicle stop rate, automobile traveler perception score, pedestrian travel speed, pedestrian space, pedestrian level-of-service (LOS) score, bicycle travel speed, bicycle LOS score, transit vehicle travel speed, and transit passenger LOS score.

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

CHAPTER ORGANIZATION

Section 2 of this chapter presents concepts used to describe urban street facility performance from an operations perspective. A multimodal evaluation framework is initially discussed. Guidance is then provided for establishing the facility analysis boundaries and the analysis period duration. A discussion about how an urban street facility is defined for the purpose of this chapter follows.

Finally, the service measures and LOS thresholds used in the methodology are examined.

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

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

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

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

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

RELATED HCM CONTENT

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

- Chapter 17, Urban Street Reliability and ATDM, which provides a methodology for evaluating travel time reliability and guidance for using this methodology to evaluate alternative active traffic and demand management (ATDM) strategies;
- Chapter 18, Urban Street Segments, which describes concepts and methodologies for the evaluation of an urban street segment;
- Chapter 29, Urban Street Facilities: Supplemental, which provides details of the reliability methodology, a procedure for sustained spillback analysis, information about the use of alternative evaluation tools, and example problems demonstrating both the urban street facility methodologies and the reliability methodology;
- Chapter 30, Urban Street Segments: Supplemental, which describes procedures for predicting platoon flow, spillback, and delay due to turns from the major street; a planning-level analysis application; and example problems demonstrating the urban street segment methodologies; and
- Section K, Urban Streets, in Part 2 of the *Planning and Preliminary Engineering Applications Guide to the HCM*, which describes how to incorporate this chapter's methods and performance measures into a planning or preliminary engineering effort.

2. CONCEPTS

This section presents concepts used to describe urban street facility performance from an operations perspective. The first subsection describes a multimodal evaluation framework that promotes consideration of each urban street travel mode and its interactions with other modes. The second assists the analyst in determining the type of analysis to be conducted. The third provides guidance for establishing the facility analysis boundaries and the analysis period duration. The fourth describes how an urban street facility is defined in terms of points, links, and segments. The fifth discusses the service measures and LOS thresholds used in the methodology. The last identifies the scope of the collective set of methodologies.

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 perception of quality of service provided to these travelers vary by mode because each mode has a different trip purpose, length, and expectation.

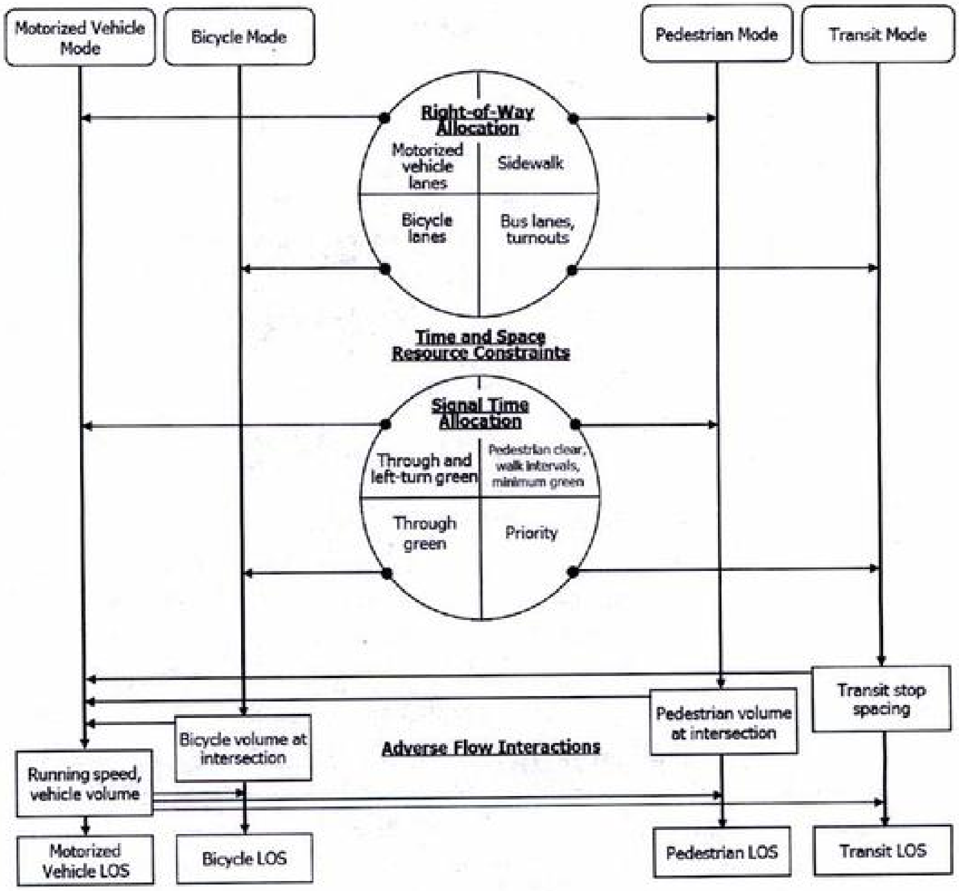
The shared street right-of-way typically requires that the modes operate in close proximity to each other, sometimes even sharing the same portion of the cross section (e.g., a vehicular traffic lane). This sharing may be workable when the modes are characterized by low demand volumes; however, acceptable 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 methodologies described in this chapter 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 16-1.

The framework shown in Exhibit 16-1 illustrates the integrated multimodal evaluation approach supported by the methodologies. 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 is based on the recognition that trip purpose, length, and expectation for each mode are different and that their combination does not produce a meaningful result.

Exhibit 16-1 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.

Exhibit 16-1
Integrated Multimodal Evaluation Framework



The lower part of Exhibit 16-1 illustrates the potential adverse interactions between the motorized vehicle mode and the other modes. As the volume or speed of the vehicle traffic stream increases, the LOS for the other modes may decrease. If bicycle, pedestrian, or transit flows increase, the LOS for the motorized vehicle 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.

ANALYSIS TYPE

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

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

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

SPATIAL AND TEMPORAL LIMITS

Analysis Boundaries

The methodologies described in this chapter are typically used to evaluate an entire facility; however, for some specific conditions, evaluation of the entire facility may not be necessary. For these conditions, the appropriate segment or intersection chapter methodology may be used alone to evaluate selected segments or intersections. In general, the analyst determines the spatial scope of each analysis (e.g., one intersection, one segment, two segments, or all segments on the facility) on the basis of analysis objectives and agency directives.

Evaluation of an individual segment or intersection may be acceptable 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 no platoon pattern is discernible 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, Exhibit 16-2 can be used to obtain an indication of whether a segment or an intersection is sufficiently distant from an upstream signal. If the distance between signals is above the trend line, the subject intersection or segment is likely to operate as effectively isolated (provided that it is not coordinated with the upstream signal).

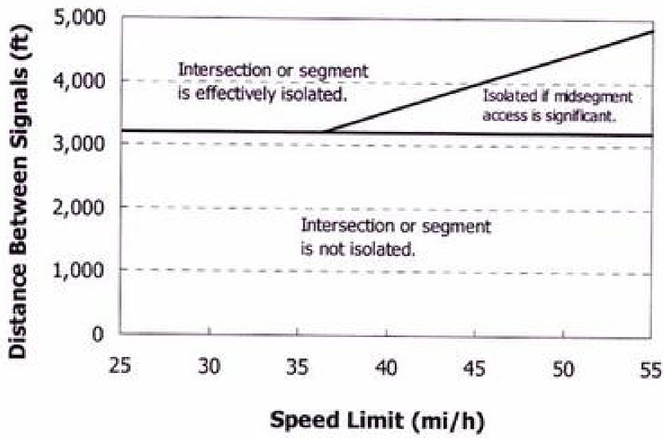


Exhibit 16-2
Signal Spacing Associated with Effectively Isolated Operation

Study Period and Analysis Period

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

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

URBAN STREET FACILITY DEFINED

Terminology

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

Facility Length Considerations

At least one intersection (or ramp terminal) along the facility must have a type of control that can impose a legal requirement to stop or yield on the through movement. A significant change in one or more facility characteristics may indicate the end of one facility and the start of a second. 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.

LOS CRITERIA

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

Motorized Vehicle Mode

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.

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 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 80% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

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

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

LOS 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 80% of the base free-flow speed, and the volume-to-capacity ratio is no greater than 1.0.

LOS C describes stable operation. The ability to maneuver and change lanes at midsegment locations may be more restricted than at LOS B. Longer queues at the boundary intersections 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 intersections. 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 intersections. 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 intersections, 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 16-3 lists the LOS thresholds established for the motorized vehicle mode on urban streets. The threshold value is interpolated when the base free-flow speed is between the values shown in the column headings of this exhibit. For example, the LOS A threshold for a segment with a base free-flow speed of 42 mi/h is 34 mi/h [= (42 – 40)/(45 – 40) × (36 – 32) + 32].

Exhibit 16-3
LOS Criteria: Motorized Vehicle Mode

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

Note: [a] 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.

Nonmotorized Vehicle 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 considered basic descriptors of the urban street character (e.g., sidewalk width). The methodologies in Chapter 18, Urban Street Segments, and Chapter 19, Signalized Intersections, mathematically combine 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 16-4 lists the range of scores associated with each LOS for the pedestrian travel mode. The LOS for this 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 16-4
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. Chapter 4 describes the concept of "cross flow" and situations where it should be considered.

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 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 16-5
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

SCOPE OF THE METHODOLOGIES

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

- *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 motorized vehicle methodology can also be used to evaluate performance with all-way STOP- or YIELD-controlled (e.g., roundabout) boundary intersections.
- *Street types.* The four methodologies were developed with a focus on arterial and collector street conditions. If a methodology is used to evaluate a local street, the performance estimates should be carefully reviewed for accuracy.
- *Flow conditions.* The four methodologies are based on the analysis of steady traffic conditions and are not well suited to the evaluation of unsteady conditions (e.g., congestion, cyclic spillback, signal preemption).
- *Target road users.* Collectively, the four methodologies were developed to estimate the LOS perceived by motorized vehicle drivers, pedestrians, bicyclists, and transit passengers. They were not developed to provide an estimate of the LOS perceived by other road users (e.g., commercial vehicle drivers, automobile passengers, delivery truck drivers, or recreational vehicle drivers). However, the perceptions of these other road users are likely to be reasonably well represented by the road users for whom the methodologies were developed.
- *Influences in the right-of-way.* A road user's perception of quality of service is influenced by many factors inside and outside of the urban street right-of-way. However, the methodologies in this chapter were specifically constructed to exclude factors that are outside of the right-of-way (e.g., buildings, parking lots, scenery, 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.

LIMITATIONS OF THE METHODOLOGIES

The urban street facility methodology uses the performance measures estimated by the segment and intersection methodologies in Chapters 18 to 23. Thus, it incorporates the limitations of these methodologies (which are identified in the respective segment or intersection chapter).

3. MOTORIZED VEHICLE METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to motorized vehicles on an urban street facility. Extensions to this methodology for evaluating more complex urban street operational elements are described in Chapter 29, Urban Street Facilities: Supplemental.

SCOPE OF THE METHODOLOGY

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

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

Spatial and Temporal Limits

Analysis Boundaries

A facility evaluation considers both directions of travel (when the street serves two-way traffic).

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

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

Study Period and Analysis Period

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

Exhibit 16-6 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 approach that is recommended.

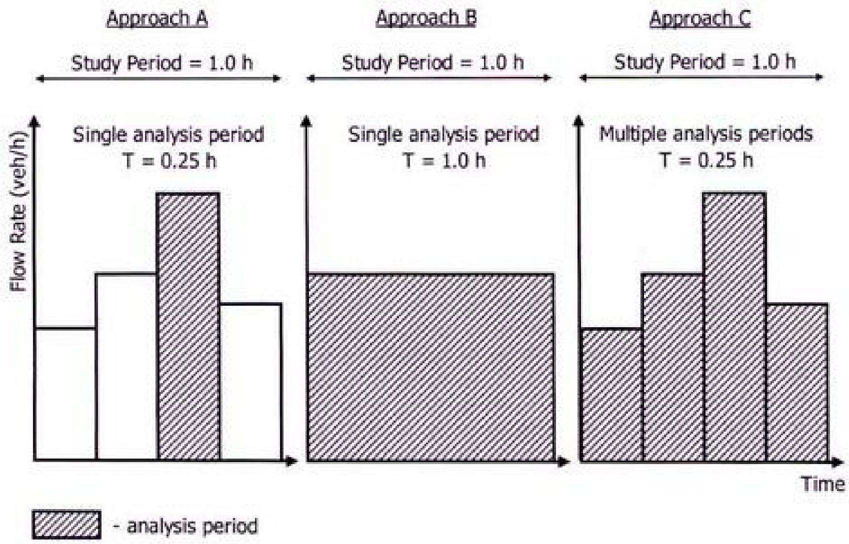


Exhibit 16-6
Three Alternative Study Approaches

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

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

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

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

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

Multiple Time Period Analysis

If the analysis period's demand volume exceeds capacity, a multiple time period analysis should be undertaken in which the study period includes an initial analysis period with no initial queue and a final analysis period with no residual queue. On a movement-by-movement and intersection-by-intersection basis, the initial queue for the second and subsequent periods is equal to the residual queue from the previous period. This approach provides a more accurate estimate of the delay associated with the congestion.

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

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. The methodology described in this chapter is focused on the evaluation of mobility on streets with these characteristics. Streets with shorter length may be evaluated with this methodology; however, the primary function of these streets is likely to be access to adjacent properties (as opposed to mobility).

Segment Length Considerations

The motorized vehicle methodology described in this section is not appropriate for the analysis of "short" segments that are bounded by signalized intersections. In contrast, the methodology described in Chapter 23, Ramp Terminals and Alternative Intersections, is appropriate for the analysis of short segments at signalized interchanges.

A short segment can experience "cyclic spillback." This spillback occurs when a queue extends back from one intersection into the other intersection (i.e., spills back) during a portion of each signal cycle and then subsides. Specific conditions under which a segment bounded by signalized intersections should be considered "short" are difficult to define. General guidance in this regard is provided in a similarly titled section in Chapter 18, Urban Street Segments.

The methodology described in this section is applicable to facilities made up of segments with each segment being 2 mi or less in length. This restriction is based on the fact that STOP-, YIELD-, or signal-controlled intersections are likely to have negligible effects on urban street operation when segment length exceeds 2 mi. Therefore, if a segment exceeds 2 mi in length, the analyst should evaluate the segment as an uninterrupted-flow highway segment with isolated intersections.

Performance Measures

Performance measures applicable to the motorized vehicle travel mode include travel speed and stop rate. LOS is also considered a performance measure. It is useful for describing street performance to elected officials, policy makers, administrators, or the public. LOS is based on travel speed and the critical volume-to-capacity ratio.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the motorized vehicle methodology. The data are listed in Exhibit 16-7 and are identified as "input data elements." They must be separately specified for each segment and for the through-movement group at each boundary intersection.

Exhibit 16-7
Required Input Data and Potential Data Sources for Motorized Vehicle Analysis

Data Category	Location	Input Data Element	Basis	Potential Data Source(s)
Geometric design	Segment	Segment length	Segment	Field data, aerial photo
Other	Segment	Analysis period duration	Facility	Set by analyst
Performance measures	Boundary intersection	Volume-to-capacity ratio	Through-movement group	HCM method output
	Segment	Base free-flow speed	Segment	HCM method output
		Travel speed	Segment	HCM method output

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.

The next-to-last column in Exhibit 16-7 indicates whether the input data are needed for a movement group at a boundary intersection, the segment, or the facility. The input data needed to evaluate the segment are identified in Chapter 18, Urban Street Segments. Similarly, the input data needed to evaluate the boundary intersections are identified in the appropriate chapter (i.e., Chapters 19 to 23).

Segment Length

Segment length is the distance between the boundary intersections that define the segment. The point of measurement at each intersection is the stop line, the yield line, or the functional equivalent in the subject direction of travel. This length is measured along the centerline of the street. If it differs in the two travel directions, an average length is used. 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.

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

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

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

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:

Equation 16-3

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

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

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), and $H_{seg,i}$ is the spatial stop rate for segment i (stops/mi).

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, Section 3, of Chapter 18 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 Motorized Vehicle LOS

LOS is determined for both directions of travel along the facility. Exhibit 16-3 lists the LOS thresholds established for this purpose. As indicated in this exhibit, LOS is defined by travel speed, where the LOS travel speed thresholds vary by 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 16-3 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.

4. PEDESTRIAN METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

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

Spatial Limits

Side of Street to Be Evaluated

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

Segment-Based Evaluation

The pedestrian methodology aggregates the performance of the segments that make up the facility. In this regard, it considers the performance of each link and boundary intersection. The methodologies for evaluating the link and boundary intersection are described in Chapters 18 and 19, respectively.

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.

Performance Measures

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

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

"Pedestrian space" is the average amount of sidewalk area available to each pedestrian walking along the facility. A larger area is more desirable from the pedestrian perspective. Exhibit 16-9 provides a qualitative description of pedestrian space that can be used to evaluate sidewalk performance from a circulation-area perspective.

Exhibit 16-9
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 16-9 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.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the pedestrian methodology. The data are listed in Exhibit 16-10 and are identified as "input data elements." They must be separately specified for each segment and direction of travel on the facility. Segment length is defined in the subsection titled Required Data and Sources in Section 3.

Exhibit 16-10
Required Input Data and Potential Data Sources for Pedestrian Analysis

Data Category	Location	Input Data Element	Potential Data Source(s)
Geometric design	Segment	Segment length	Field data, aerial photo
		Presence of a sidewalk	Field data, aerial photo
Performance measures	Segment	Pedestrian space	HCM method output
		Pedestrian travel speed	HCM method output
		Pedestrian LOS score for segment	HCM method output

Presence of a Sidewalk

A sidewalk is a paved walkway that is provided at the side of the roadway. Pedestrians are assumed to walk in the street if a sidewalk is not present. 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 18 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 is 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 18. 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 described in Chapter 18. One score is needed for each direction of travel of interest for each segment on the facility.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to pedestrians. The methodology is applied through a series of four steps that culminate in the determination of the facility LOS. These steps are illustrated in Exhibit 16-11.

Exhibit 16-11
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

A methodology for evaluating off-street pedestrian facilities is provided in Chapter 24, Off-Street Pedestrian and Bicycle Facilities. Off-street facilities include those for which the characteristics of motor vehicle traffic do not play a strong role in determining quality of service from the perspective of pedestrians.

COMPUTATIONAL STEPS

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 18, 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. The analyst should verify that the intersection corners and crosswalks adequately accommodate pedestrians by using the methodology in Section 5 of Chapter 19.

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 18. 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), and $S_{Tp,seg,i}$ is the travel speed of through pedestrians for segment i (ft/s).

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 is a travel-time-weighted average of the pedestrian LOS scores for the individual links and intersections that make up the facility. The LOS score for each segment (determined by using the procedures described in Chapter 18) is used to compute the total weighted score for the facility. The total travel time for the facility is computed by using the travel speed from Step 2. The LOS score for the facility is computed by using the ratio of the total weighted score and the total travel time. This score is calculated by using Equation 16-7:

Equation 16-7

$$I_{p,F} = 0.75\left[\frac{\sum_{i=1}^{m} WTT_{p,i}}{\left(\frac{\sum_{i=1}^{m} L_i}{S_{Tp,F}}\right)}\right]^{\frac{1}{3}} + 0.125$$

with

$$WTT_{p,i} = \left(\frac{L_i}{S_{Tp,seg,i}}\right)\left(\frac{I_{p,seg,i} - 0.125}{0.75}\right)^3$$

Equation 16-8

where

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

$WTT_{p,i}$ = travel-time-weighted average pedestrian LOS score for segment i, and

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

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 16-4 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, LOS is determined by using Exhibit 16-5 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.

5. BICYCLE METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

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

Spatial Limits

Travel Directions to Be Evaluated

Urban street 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 bicycle is assumed to travel in the street (possibly in a bicycle lane) and in the same direction as adjacent motorized vehicles.

Segment-Based Evaluation

The bicycle methodology aggregates the performance of the segments that make up the facility. In this regard, it considers the performance of each link and boundary intersection. The methodologies for evaluating the link and boundary intersection are described in Chapters 18 and 19, respectively.

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.

Performance Measures

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

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

Limitations of the Methodology

This subsection identifies a known limitation of the bicycle methodology. Specifically, the methodology is not applicable when the bicycle lanes occur intermittently along the facility. If this condition is present, the analyst should consider using alternative methods or tools for the evaluation.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the bicycle methodology. The data are listed in Exhibit 16-12 and are identified as "input data elements." They must be separately specified for each segment and direction of travel on the facility. Segment length is defined in the subsection titled Required Data and Sources in Section 3.

Exhibit 16-12
Required Input Data and Potential Data Sources for Bicycle Analysis

Data Category	Location	Input Data Element	Potential Data Source(s)
Geometric design	Segment	Segment length	Field data, aerial photo
Performance measures	Segment	Bicycle travel speed	HCM method output
		Bicycle LOS score for segment	HCM method output

Bicycle Travel Speed

Bicycle travel speed is 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 18. 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 18. One score is needed for each direction of travel of interest for each segment on the facility.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to bicyclists. The methodology is applied through a series of three steps that culminate in the determination of the facility LOS. These steps are illustrated in Exhibit 16-13.

Exhibit 16-13
Bicycle Methodology for Urban Street Facilities

Step 1: Determine Bicycle Travel Speed

↓

Step 2: Determine Bicycle LOS Score

↓

Step 3: Determine Bicycle LOS

A methodology for evaluating off-street bicycle facilities is provided in Chapter 24, Off-Street Pedestrian and Bicycle Facilities. Off-street facilities include those for which the characteristics of motor vehicle traffic do not play a strong role in determining quality of service from the perspective of bicyclists.

COMPUTATIONAL STEPS

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 18. The bicycle travel speed for the facility is calculated by using Equation 16-9:

Equation 16-9

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

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 is a travel-time-weighted average of the bicycle LOS scores for the individual links and intersections that make up the facility. The LOS score for each segment (determined by using the procedures described in Chapter 18) is used to compute the total weighted score for the facility. The total travel time for the facility is computed by using the travel speed from Step 1. The LOS score for the facility is computed by using the ratio of the total weighted score and the total travel time. This score is calculated by using Equation 16-10:

Equation 16-10

$$I_{b,F} = 0.75 \left[\frac{\sum_{i=1}^{m} WTT_{b,i}}{\left(\frac{\sum_{i=1}^{m} L_i}{S_{Tb,F}} \right)} \right]^{\frac{1}{3}} + 0.125$$

with

Equation 16-11

$$WTT_{b,i} = \left(\frac{L_i}{S_{Tb,seg,i}} \right) \left(\frac{I_{b,seg,i} - 0.125}{0.75} \right)^3$$

where $I_{b,F}$ is the bicycle LOS score for the facility, $WTT_{b,i}$ is the travel-time-weighted average bicycle LOS score for segment i, and $I_{b,seg,i}$ is the bicycle LOS score for segment i.

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 16-5 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.

6. TRANSIT METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

Spatial Limits

Travel Directions to Be Evaluated

Urban street 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.*

Route-Based Evaluation

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

Performance Measures

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

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

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the transit methodology. The data are listed in Exhibit 16-14 and are identified as "input data elements." They must be separately specified for each segment and direction of travel on the facility. Segment length is defined in the Required Data and Sources subsection in Section 3.

Exhibit 16-14
Required Input Data and Potential Data Sources for Transit Analysis

Data Category	Location	Input Data Element	Potential Data Source(s)
Geometric design	Segment	Segment length	Field data, aerial photo
Performance measures	Segment	Transit travel speed	HCM method output
		Transit LOS score for segment	HCM method output

Transit Travel Speed

Transit travel speed is 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 18. 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 18. One score is needed for each direction of travel of interest for each segment on the facility.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of an urban street facility in terms of its service to transit passengers. The methodology is applied through a series of three steps that culminate in the determination of facility LOS. These steps are illustrated in Exhibit 16-15.

Exhibit 16-15
Transit Methodology for Urban Street Facilities

Step 1: Determine Transit Travel Speed

↓

Step 2: Determine Transit LOS Score

↓

Step 3: Determine Transit LOS

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).

COMPUTATIONAL STEPS

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 18. The transit travel speed for the facility is calculated by using Equation 16-12:

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

Equation 16-12

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 is 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 18. The score for the facility is calculated by using Equation 16-13:

Equation 16-13

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

where $I_{t,F}$ is the transit LOS score for the facility, and $I_{t,seg,i}$ is the transit LOS score for segment i.

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 16-5 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.

7. APPLICATIONS

EXAMPLE PROBLEMS

Chapter 29, Urban Street Facilities: Supplemental, describes the application of each of the four methodologies through the use of example problems. The focus of the examples is on illustrating the multimodal facility evaluation process. An operational analysis level is used for all examples.

GENERALIZED DAILY SERVICE VOLUMES

Generalized daily service volume tables provide a means of assessing 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 semiactuated, 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.
- 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 16-16 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 16-16
Generalized Daily Service Volumes for Urban Street Facilities

K-Factor	*D*-Factor	Daily Service Volume by Lanes, LOS, and Speed (1,000 veh/day)											
		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	1.7	11.8	17.8	NA	2.2	24.7	35.8	NA	2.6	38.7	54.0
	0.60	NA	1.6	10.8	16.4	NA	2.0	22.7	32.8	NA	2.4	35.6	49.5
0.10	0.55	NA	1.6	10.7	16.1	NA	2.0	22.3	32.2	NA	2.4	34.9	48.6
	0.60	NA	1.4	9.8	14.7	NA	1.8	20.4	29.5	NA	2.2	32.0	44.5
0.11	0.55	NA	1.4	9.7	14.6	NA	1.8	20.3	29.3	NA	2.1	31.7	44.1
	0.60	NA	1.3	8.9	13.4	NA	1.7	18.6	26.9	NA	2.0	29.1	40.5
Posted Speed = 45 mi/h													
0.09	0.55	NA	7.7	15.9	18.3	NA	16.5	33.6	36.8	NA	25.4	51.7	55.3
	0.60	NA	7.1	14.5	16.8	NA	15.1	30.8	33.7	NA	23.4	47.4	50.7
0.10	0.55	NA	7.0	14.3	16.5	NA	14.9	30.2	33.1	NA	23.0	46.5	49.7
	0.60	NA	6.4	13.1	15.1	NA	13.6	27.7	30.3	NA	21.0	42.7	45.6
0.11	0.55	NA	6.3	13.0	15.0	NA	13.5	27.5	30.1	NA	20.9	42.3	45.2
	0.60	NA	5.8	11.9	13.8	NA	12.4	25.2	27.6	NA	19.1	38.8	41.5

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, semiactuated 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 16-16 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 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 in determining where improvements might be needed. However, any urban street identified as likely to experience problems or need improvement should 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. The values used for the facilities under study should be reasonable. Also, if any characteristic is significantly different from the typical values used to develop Exhibit 16-16, 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.

ANALYSIS TYPE

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

Operational Analysis

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

Design Analysis

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

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

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

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

Planning and Preliminary Engineering Analysis

The objective of a planning and preliminary engineering analysis can be (*a*) to determine the LOS for either a proposed facility or an existing facility in a future year or (*b*) 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 because 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 18 to 23.

USE OF ALTERNATIVE TOOLS

Chapter 29, Urban Street Facilities: Supplemental, includes a set of examples illustrating the use of alternative tools in addressing the stated limitations of this chapter and Chapter 18, 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 midsegment parking maneuvers on facility operation, (*c*) the effect of using a roundabout as a segment boundary, 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.

8. REFERENCES

These references can be found 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.; Parsons Brinckerhoff; KFH Group, Inc.; Texas A&M Transportation Institute; and Arup. *TCRP Report 165: Transit Capacity and Quality of Service Manual,* 3rd ed. Transportation Research Board of the National Academies, Washington, D.C., 2013.

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

This chapter describes a methodology for evaluating the travel time reliability experienced by motorists on an urban street facility. Travel time reliability reflects the distribution of trip travel time over an extended period. The distribution arises from the occurrence of a number of factors that influence travel time (e.g., weather events, incidents, work zone presence). The distribution describes *how often* these factors occur and *how bad* operations are as a result.

The methodology's reliability performance measures can be used for a variety of facility management functions. For example, they can be used as the basis for quantifying the degree of severity of Level of Service (LOS) F (oversaturated) conditions. They can also be used for developing agency performance standards for oversaturated facilities. Finally, they can be used to quantify the impacts of physical and operational treatments designed to improve travel time reliability.

The methodology is focused on the evaluation of an urban street facility (with consideration of the segments that make it up). It is used with the methodologies in other *Highway Capacity Manual* (HCM) chapters to compute the desired performance measures. Specifically, the methodology for aggregating segment performance measures to obtain an estimate of facility performance is described in Chapter 16. The methodology for evaluating the individual segments is described in Chapter 18. The methodologies in Chapters 16 and 18 are applicable to an urban street facility that typically has a length of 1 mi or more in downtown areas and 2 mi or more in other areas.

The methodology described in this chapter is largely based on the product of a Strategic Highway Research Program 2 project (*1*). Contributions to the methodology from other research are referenced in the relevant sections.

An important application of the methodology is in the evaluation of active traffic and demand management (ATDM) tactics. An ATDM tactic adapts the facility configuration and controls to (or in anticipation of) variations in demand, incidents, and weather to maintain a high level of facility performance (*2*). These tactics are related to temporal changes in speed and signal control, geometric configuration, and traffic demand volume. In its current form, the methodology is most amenable to the evaluation of geometric configuration modifications (e.g., dynamic lane assignments, reversible lanes, and dynamic turn restrictions).

CHAPTER ORGANIZATION

Section 2 presents travel time variability and reliability concepts, including objectives of reliability analysis and definitions of reliability terms. This section also includes an overview of ATDM and the range of strategies applicable to urban street facilities.

Section 3 presents the core methodology for evaluating urban street reliability. It includes a description of the scope of the methodology and its required input data. It concludes with an overview of the reliability evaluation methodology.

Section 4 describes various ATDM strategies applicable to urban streets and typical tactics for implementing each strategy.

Section 5 presents guidance on using the results of the reliability evaluation. It includes example results that illustrate an application of the reliability methods to an urban street facility, and it discusses typical cases for which a reliability evaluation can provide useful information.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 16, Urban Street Facilities, which describes concepts and methodologies for the evaluation of an urban street facility;
- Chapter 18, Urban Street Segments, which describes concepts and methodologies for the evaluation of an urban street segment;
- Chapter 29, Urban Street Facilities: Supplemental, which provides details of the reliability methodology, a procedure for sustained spillback analysis, information about the use of alternative evaluation tools, and example problems demonstrating both the urban street facility methodologies and the reliability methodology;
- Chapter 30, Urban Street Segments: Supplemental, which describes procedures for predicting platoon flow, spillback, and delay due to turns from the major street; a planning-level analysis application; and example problems demonstrating the urban street segment methodologies; and
- Chapter 37, ATDM: Supplemental, which summarizes the steps involved in designing an ATDM program.

2. CONCEPTS

This section summarizes key reliability concepts. The first subsection discusses the reasons why an analyst might want to evaluate a facility's reliability. The second provides important definitions related to reliability evaluation. The third describes performance measures and typical values. The fourth summarizes key ATDM concepts.

OBJECTIVES FOR RELIABILITY ANALYSIS

An important step in any analysis is defining why the analysis is being performed. Key questions or issues should be defined, performance measures that help answer those questions identified, and a basis of comparison for interpreting the analysis results established. Reliability analysis is no different. The following are examples of potential objectives of a reliability analysis:

- Tracking the reliability of a set of facilities in a jurisdiction or region over time to prioritize them for operational or physical treatments,
- Diagnosing the primary causes of the reliability problems on a given facility so that an improvement program can be developed, and
- Evaluating the effects of a particular treatment or improvement on facility reliability.

Reliability analysis can be used to improve the operation, planning, prioritization, and programming of transportation system improvement projects.

More broadly, travel time reliability analysis can be used to improve the operation, planning, prioritization, and programming of transportation system improvement projects in the following applications: long-range transportation plans, transportation improvement programs, corridor or areawide plans, major investment studies, congestion management, operations planning, and demand forecasting. The Use Cases portion of Section 5, Applications, describes these applications in greater detail.

DEFINITIONS

The following terms are used in this chapter:

- **Scenario.** A unique combination of traffic demand, capacity, geometry, and traffic control conditions. It can represent one or more analysis periods provided that all periods have the same combination of demand, capacity, geometry, and control.
- **Study period.** The time interval (within a day) that is represented by the performance evaluation. It consists of one or more consecutive analysis periods.
- **Analysis period.** The time interval evaluated by a single application of an HCM methodology.
- **Study section.** The length of facility over which reliability is to be computed. Since reliability is computed for through traffic only, the length of the facility should not be so long that through traffic is a low percentage of total traffic on the facility. The length of facility to be

evaluated should be less than the distance a vehicle traveling at the average speed can achieve in 15 min.

- **Reliability reporting period.** The specific days over which reliability is to be computed, for example, all weekdays in a year.
- **Special event.** Short-term events that produce intense traffic demands on a facility for limited periods of time. These demands may be addressed by temporary changes in the facility's geometry or traffic control characteristics, or both. Example special events include major sporting events, concerts, and festivals.

Additional terms are defined in Chapter 9, Glossary.

ACTIVE TRAFFIC AND DEMAND MANAGEMENT

ATDM is dynamic real-time management and control of the arterial system.

ATDM is the dynamic management, control, and influence of travel demand, traffic demand, and traffic flow of transportation facilities. Through the use of available tools and assets, traffic flow is managed and traveler behavior is influenced in real time to achieve operational objectives, such as preventing or delaying breakdown conditions, improving safety, promoting sustainable travel modes, reducing emissions, or maximizing system efficiency (3).

Spectrum of ATDM Applications

ATDM applications on urban streets can range from the simple to the complex.

ATDM applications on an urban street can range widely from innovative uses of conventional control hardware to the deployment of state-of-the-art real-time traffic control systems. A conventional traffic-actuated signal that measures detector occupancy in real time and uses the information to adjust phase splits automatically (within a fixed narrow allowed range) and skip unneeded phases is an example of a relatively simple ATDM application. An example of a more advanced ATDM application is traffic adaptive control, under which the system controller dynamically measures demand and adjusts phase splits and offsets in real time to serve platoons of vehicles.

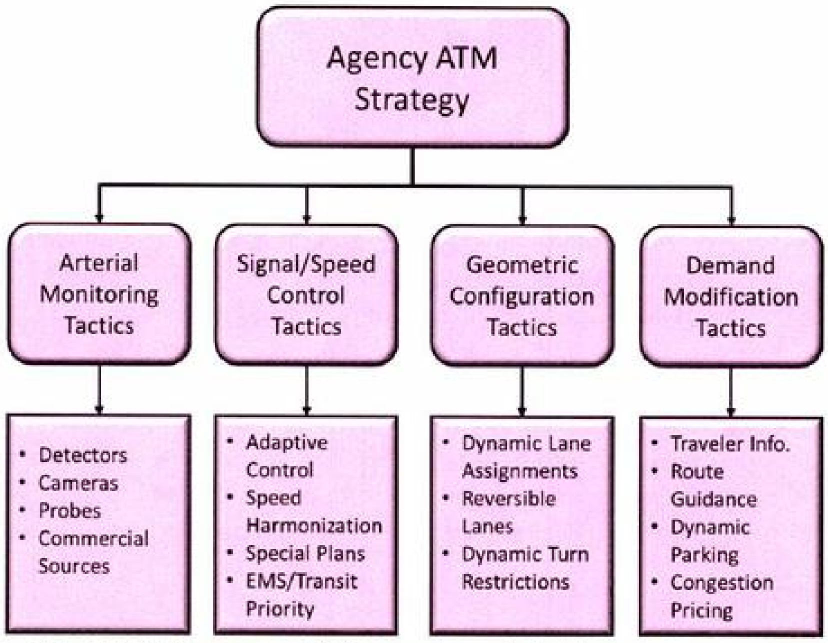
ATDM Options for Urban Streets

When an agency adopts an overall ATDM strategy for managing its urban streets, the strategy is typically composed of various tactical actions taken from one or more of the following tactical groups:

- Arterial monitoring tactics,
- Signal and speed control tactics,
- Geometric configuration tactics, and
- Demand modification tactics.

Each tactical group has a specific objective and set of measures that are implemented to achieve the overall agency ATDM goals. Typical ATDM measures associated with each tactical group are shown in Exhibit 17-1. These groups and measures are reviewed in the following paragraphs.

Exhibit 17-1
ATDM Tactics and Measures for Urban Streets



Note: ATM = active traffic management; EMS = emergency medical services.
Source: Adapted from Dowling and Elias (*2*).

Arterial Monitoring Tactics

The objective of ATDM measures within the arterial monitoring tactical group is to obtain actionable real-time information on urban street performance. This objective may be achieved by several means, such as the use of closed-circuit television cameras and vehicle detectors, communication between connected vehicles and the signal controller, or the purchase of travel speed data from a commercial vendor of real-time traffic data. The choice of measures to achieve the tactical objective of monitoring becomes the agency's monitoring tactic for the facility.

Speed and Signal Control Tactics

The objective of ATDM measures within the speed and signal control tactical group is to adapt signal timing (and speed limits if appropriate) to maximize production (capacity) and minimize cost (delay and stops). Measures to achieve this objective include dynamic signal control (traffic actuated, traffic responsive, and traffic adaptive) and dynamic speed limits that may be communicated via roadside signs or overhead signs or that may be communicated directly to connected vehicles. Signal timing affects the capacity of a street by changing the allocation of green time between users of the street (through movements, transit, pedestrians, and turn movements). It can also affect the speed of travel on the facility through signal coordination. *NCHRP Synthesis 403* (*4*) and the *Signal Timing Manual* (*5*) provide more information on advanced signal control options.

Geometric Configuration Tactics

The objective of ATDM measures within the geometric configuration tactical group is to adjust lane use on the urban street dynamically to improve the match with demand, thereby increasing the street's capacity. These measures can include changing the number of lanes designated for turns, changing the vehicle types allowed to use a lane, or even changing the direction of flow for certain lanes. Practicalities limit the ability to open and close parking lanes on a street dynamically (drivers must be warned when they first park their vehicle of when they must leave). Dynamic turn restrictions are included in this tactical approach.

Demand Management Tactics

The objective of ATDM measures within the demand modification tactical group is to improve the match between demand and the available capacity. In the context of the HCM, demand management primarily relates to traveler information services and guidance. The travel information is provided in the hope that a better-informed traveling public will shift to less congested facilities and thereby better balance demand with available capacity. A more proactive form of demand management is to provide actual dynamic route and mode of travel guidance, making travelers aware of additional routing and modal options. Congestion pricing can be used to reinforce the traveler guidance for the urban street.

3. CORE METHODOLOGY

At its core, the reliability methodology consists of hundreds of repetitions of the urban street facility methodology presented in Chapter 16. In contrast to the Chapter 16 methodology, where the inputs represent average values for a defined analysis period, the reliability methodology varies the demand, capacity, geometry, and traffic control inputs to the facility methodology with each repetition (i.e., scenario).

All the HCM performance measures output by the facility methodology are assembled for each scenario and used to describe the facility's performance over the course of a year (or other user-defined reliability reporting period). Performance can be described on the basis of a percentile result (e.g., the 80th or 95th percentile travel time) or the probability of achieving a particular level of service (e.g., the facility operates at LOS D during X% of weekday hours during the year). Many other variability and reliability performance measures can be developed from the facility's travel time distribution.

The reliability methodology is sensitive to the main sources of variability that lead to travel time unreliability. These sources are as follows:

- Temporal variability in traffic demand—both regular variations by hour of the day, day of the week, and month or season of the year and random variations between hours and days;
- Incidents that block travel lanes or that otherwise affect traffic operations and thus capacity;
- Weather events that affect capacity and possibly demand;
- Work zones that close or restrict travel lanes, thus affecting capacity; and
- Special events producing atypical traffic demands that may require management by special traffic control measures.

Work zones and special events are location-specific parameters that must be provided by the analyst. Location-specific data related to traffic demand variability, incidents, and weather patterns are best provided by the analyst if they are available; however, the reliability methodology also provides default values for use when local data are unavailable or the analysis does not require that level of precision.

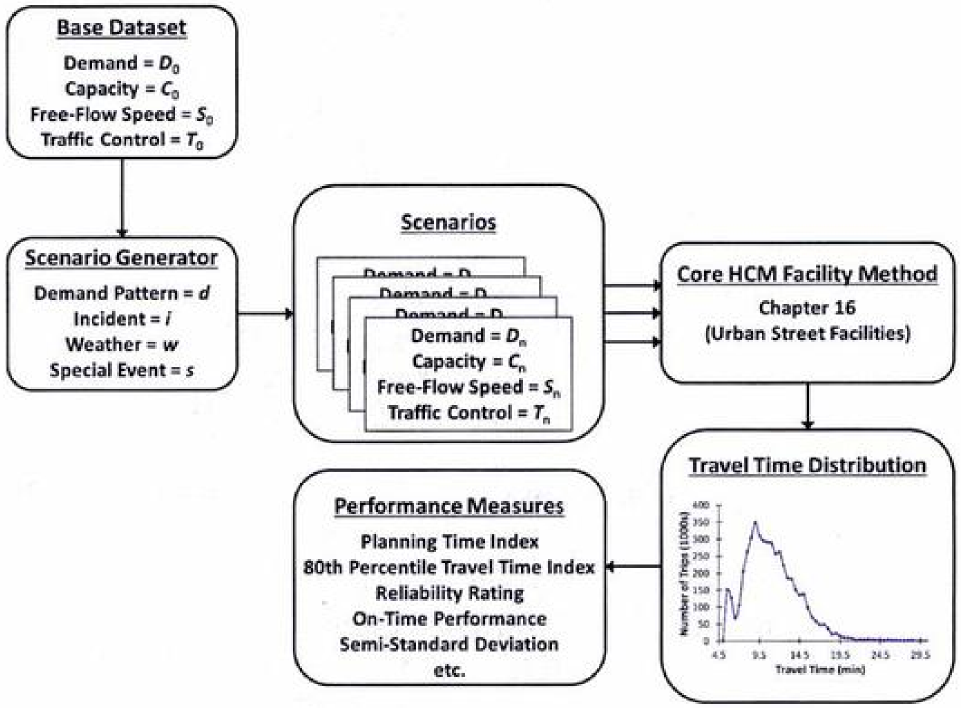
Input data needed for a reliability evaluation (beyond those needed for an HCM facility evaluation) consist of demand variation data, incident data, weather data, work zones, and special events. The first three types of data can be defaulted when they are not available locally.

Scenarios are built from combinations of conditions associated with each source of travel time variability. For example, one scenario could represent demand volumes representative of Fridays in May, fair weather, and one lane closed for 30 min because of an incident that occurs during the p.m. peak hour. A probability of occurrence is associated with each scenario on the basis of local data provided by the analyst (or the method's default data) and is used to develop a travel time distribution for the reliability reporting period.

The method for estimating the travel time distribution calculates the performance of a series of scenarios representing different combinations of conditions that affect a facility's demand or capacity, or both.

Exhibit 17-2 provides a high-level representation of the methodology for estimating the travel time distribution. The base dataset consists of all the data needed to evaluate the base HCM facility methodology for a single study period, plus data that describe the variations in demand, weather, and so forth that occur over the course of the reliability reporting period, along with the frequency of a particular event's occurrence. The scenario generator creates a set of scenarios in which the base facility demand and capacity are adjusted to reflect the changes in demand and capacity that occur under each combination of conditions. Each scenario is submitted to the HCM facility methodology, which calculates the facility travel time associated with the scenario. The individual facility travel times are then compiled into the facility's travel time distribution. This distribution can be used to develop a variety of reliability and variability performance measures for the facility.

Exhibit 17-2
High-Level Representation of the Method for Estimating the Travel Time Distribution



Because of the hundreds (or even thousands) of scenarios that are generated, implementation of this method is only practical through software. Software automates the scenario generation process, performs the computations associated with the HCM facility methodology for each scenario, and stores and processes the output performance measures generated for each scenario. Source code listings for the research-grade computational engine (i.e., STREETVAL) are provided in the Technical Reference Library in HCM Volume 4.

SCOPE OF THE METHODOLOGY

The reliability methodology can be used to evaluate the following sources of unreliable travel time:

- Demand fluctuations,
- Weather,
- Traffic incidents,
- Work zones,
- Special events,

- Inadequate base capacity, and
- Traffic control devices on urban streets.

Demand fluctuations are represented in the methodology in terms of systematic and random demand variation by hour of day, day of week, and month of year. Fluctuations due to diversion are not addressed directly by the methodology but can be optionally provided by the analyst for work zones and special events through the demand specified in an alternative dataset.

Performance Measures

The reliability methodology generates distributions of the performance measures produced by the HCM facility methodologies. Each distribution represents the variation of one performance measure during the reliability reporting period. Performance measures applicable to urban street facilities include travel time, travel speed, delay, and spatial stop rate, among others.

The distribution of a performance measure can be aggregated over the reliability reporting period to produce an overall total (or average). Measures of this nature are described in the first subsection to follow. The distribution can also be described by using percentile values to indicate measure variability or propensity for failure. Measures of this nature are described in the second subsection to follow.

Measures Describing Typical (Average) Conditions

Several traditional performance measures are used to describe the overall average operation of a facility. In combination with reliability measures, they provide a complete picture of facility performance and form a useful basis for alternative evaluation. The following are useful traditional measures:

- *Travel time* (minutes). Travel time is a versatile measure, since it can be monitored over time (for trend analysis), monetized (in calculating benefits), and used in the calculation of other measures (e.g., delay). Facility lengths usually remain the same over time, allowing apples-to-apples comparisons of travel times estimated for a facility in different years or under different circumstances.
- *Annual delay* (vehicle hours and person hours). Annual delay is the average vehicle hours of travel or person hours of travel occurring minus what would occur under free-flow conditions. Delay is useful because economic analyses have a long history of monetizing delay.

Measures Describing Reliability

Numerous performance measures are available for quantifying aspects of facility travel time reliability. Many of them combine a specified percentile value with a typical or ideal value to compute a dimensionless index that describes the relative variability, relative propensity for failure, or both. For urban street facilities, the base free-flow speed is used as the ideal value on which all speed- and travel-time-related reliability indices are based.

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

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

A commonly used index is the travel time index (TTI). It is defined as the ratio of the actual travel time on a facility to the travel time at the base free-flow speed. Other indices are defined in Section 2 of Chapter 4, Traffic Operations and Capacity Concepts.

The following measures are useful for describing (*a*) travel time variability or (*b*) the success or failure of individual trips in meeting a target travel time or speed:

- *95th percentile TTI or planning time index (PTI)* (unitless). The ratio of the 95th percentile highest travel time to the travel time at the base free-flow speed. This measure is useful for estimating how much extra time travelers must budget to ensure an on-time arrival and for describing near-worst-case conditions on urban facilities.
- *80th percentile TTI* (unitless). The ratio of the 80th percentile highest travel time to the travel time at the base free-flow speed. Research indicates that this measure is more sensitive to operational changes than the PTI (*6*), which makes it useful for comparison and prioritization purposes.
- *50th percentile TTI* (unitless). The ratio of the 50th percentile highest travel time to the travel time at the base free-flow speed. This measure can be used for trend analysis and to demonstrate changes in performance resulting from an operational strategy, capacity improvement, or change in demand.
- *Mean TTI* (unitless). The ratio of the average travel time to the travel time at the base free-flow speed. This measure can be used for the same purposes as the 50th percentile TTI. However, the mean TTI will typically have somewhat higher values than the 50th percentile TTI because of the influence of rare, very long travel times in the distribution.
- *Failure or on-time measures* (percentage). The percent of trips (or percent of time) with space mean speeds above (on time) or below (failure) one or more target values (e.g., 35, 45, and 50 mi/h). These measures address how often trips succeed or fail in achieving a desired travel time or speed.
- *Reliability rating* (percentage). The percentage of vehicle miles traveled on the facility associated with a TTI less than 2.50. This threshold approximates the point beyond which urban street facility travel times become much more variable (i.e., unreliable).
- *Semi–standard deviation* (unitless). A one-sided standard deviation, with the reference point at the base free-flow speed instead of the mean. It provides the variability distance from free-flow conditions.
- *Standard deviation* (unitless). The standard statistical measure.
- *Misery index* (unitless). This measure is useful as a descriptor of near-worst-case conditions on rural facilities.

In many cases, an analyst may wish to consider several of these measures to obtain a complete picture of travel time reliability. However, the reliability rating is recommended as part of any HCM-based reliability analysis because it is a single measure reflecting the traveler point of view (by stating the potential for unreliable travel). The use of the reliability rating and other reliability measures is illustrated in example problems in Chapter 29, Urban Street Facilities: Supplemental.

Limitations of the Methodology

Because the reliability methodology is based on applying the urban street methodologies multiple times, it inherits the limitations of those methodologies, as described in Chapters 16, 18, and 19, respectively. The reliability methodology has additional limitations as described in the following paragraphs.

In general, the urban street reliability methodology can be used to evaluate the performance of most urban street facilities. However, the methodology does not address the following events or conditions:

- Truck pickup and delivery (double parking);
- Signal malfunction;
- Railroad crossing;
- Railroad and emergency vehicle preemption;
- Signal plan transition; and
- Fog, dust storms, smoke, high winds, or sun glare.

Lane or shoulder blockage due to truck pickup-and-delivery activities in downtown urban areas can be considered incident-like in terms of the randomness of their occurrence and the temporal extent of the event. The dwell time for these activities can range from 10 to 20 min (*7*).

A signal malfunction occurs when one or more elements of the signal system are not operating in the intended manner. These elements include vehicle detectors, signal heads, and controller hardware. A failure of one or more of these elements typically results in poor facility operation.

A railroad crossing the facility at a midsegment location effectively blocks traffic flow while the train is present. Train crossing time can be lengthy (i.e., typically 5 to 10 min) and can result in considerable congestion extending for one or more subsequent analysis periods.

Railroad preemption occurs when a train crosses a cross-street leg of a signalized intersection. The signal operation is disrupted to clear the tracks safely. Signal coordination may be disrupted for several cycles after train clearance.

When a new timing plan is invoked, the controller goes through a transition from the previous plan to the new plan. The transition period can last several cycles, during which traffic progression is significantly disrupted.

Some weather conditions that restrict driver visibility or degrade vehicle stability are not addressed by the methodology. They include fog, dust storms, smoke, and high winds.

REQUIRED DATA AND SOURCES

HCM Facility Analysis Input Data

The input data needed to evaluate an urban street facility for one analysis period are also needed for a reliability evaluation. These input data are described in Chapters 16, 18, and 19 and are referred to as an *HCM dataset* in this chapter.

For some reliability evaluations, more than one HCM dataset will be required. One HCM dataset, the *base dataset*, is always required and is used to describe base conditions (particularly demand and factors influencing capacity and free-flow speed) when work zones and special events are not present. The base dataset can represent average demand conditions or the demand measured on a specific day. The reliability methodology includes a procedure for factoring the average-day or specific-day demands (on the basis of user-supplied or defaulted demand patterns) to generate demands representative of all other time periods during the reliability reporting period.

Additional HCM datasets are used, as needed, to describe conditions when a specific work zone is present or when a special event occurs. They are called *alternative datasets*. The user must specify any changes in base conditions (e.g., demand, traffic control, available lanes) associated with the work zone or special event, along with the times when the alternative dataset is in effect. For example, if a work zone exists during a given month, an alternative dataset is used to describe average conditions for the analysis period during that month.

Additional Data Required for Reliability Evaluation

Additional data (beyond those described in the previous subsection) are required for a reliability evaluation. Exhibit 17-3 gives the general categories of data that are required.

Exhibit 17-3
General Data Categories Required for a Reliability Evaluation

Data Category	Description
Functional class	Functional class required when defaulted demand patterns are used.
Nearest city	Required when defaulted weather data are used.
Geometrics	Presence of shoulder.
Time periods	Analysis period, study period, reliability reporting period.
Demand patterns	Hour-of-day (K), day-of-week, and month-of-year demand factors relative to annual average daily traffic. Demand change due to rain and snow. Can be defaulted.
Weather	Rain, snow, and temperature data by month. Pavement runoff duration for a snow event. Can be defaulted.
Incidents	Probabilities of specific crash and incident types by location. Alternatively, segment and intersection crash frequencies. Crash frequency adjustment factors. Factors influencing incident duration. The latter two factors can be defaulted.
Work zones and special events	Changes to base conditions (alternative dataset) and schedule.
Traffic counts	Day and time of traffic counts used in base and alternative datasets.

As shown in Exhibit 17-3, most reliability-specific inputs can be defaulted. Default values are identified in the following subsections. They allow analysts in "data poor" regions lacking detailed demand, weather, or incident data to apply the reliability methodology and obtain reasonable results. At the same time, the methodology allows analysts in "data rich" regions to provide local data for these inputs when the most accurate results are desired.

Functional Class

The functional class of the subject facility is a required input when the analyst chooses to use the default time period adjustment factors. These factors are used for estimating traffic volume during each of the various scenarios that make up the reliability reporting period. The default factors are described in the subsequent section titled Demand Pattern Data.

The following functional classes are considered:

- Urban expressway,
- Urban principal arterial street, and
- Urban minor arterial street.

An urban principal arterial street emphasizes mobility over access. It serves intra-area travel, such as that between a central business district and outlying residential areas or that between a freeway and an important activity center. It is typically used for relatively long trips within the urban area or for through trips that enter, leave, or pass through the city. An urban minor arterial street provides a balance between mobility and access. It interconnects with and augments the urban principal arterial street system. It is typically used for trips of moderate length within relatively small geographic areas (*8*).

The methodology addresses roadways that (*a*) belong to one of the aforementioned classes and (*b*) do not have full access control. If a roadway has full access control, it is considered to be a freeway, and the analyst should use the freeway methodology.

Nearest City

The nearest city is a required input when the analyst chooses to use the default weather data. The analyst selects from 284 U.S. cities. The default weather data are described in a subsequent subsection titled Weather Data.

Geometrics

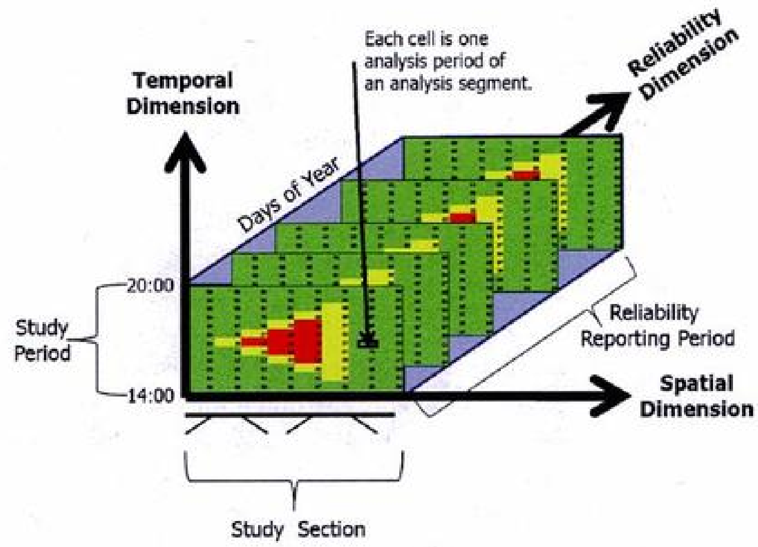
The indication of the presence of outside (i.e., right-side) shoulders is a required input when the analyst chooses to use the default incident location data. This input is specified for the facility. The default incident location data are described in a subsequent subsection titled Incident Data.

For a shoulder to be considered present, it must be wide enough to store a disabled vehicle (so that the vehicle does not block traffic flow in the adjacent traffic lane). If on-street parking is allowed, the analyst will need to determine whether occupancy of the shoulder during the study period is sufficient to preclude its use as a refuge for disabled vehicles. The proportion of on-street parking occupied would need to be less than 30% to provide reasonable assurance of the opportunity to move a disabled vehicle from the through lanes to an open stall.

Time Periods

The time periods that need to be specified include the analysis period, the study period, and the reliability reporting period. Exhibit 17-4 presents the relationships between these periods. They are defined in the following paragraphs.

Exhibit 17-4
Temporal and Spatial Dimensions of Reliability



Source: Zegeer et al. (*1*).

Analysis Period

The analysis period is the time interval used for the performance evaluation. It can range from 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. A shorter duration in this range is typically used for operational analyses. Additional guidance for determining the analysis period duration is provided in Chapter 16, Urban Street Facilities.

Shorter analysis periods allow relatively brief incidents and weather events to be considered in reliability evaluations.

A shorter analysis period duration is desirable for reliability evaluations because it reduces the minimum event duration threshold and thereby increases the number of incidents and weather events that are included in scenarios. In this regard, the structure of the reliability methodology is such that events that are shorter than one-half of the analysis period duration are ignored (i.e., they will not be recognized in the scenario generation process).

Study Period

If an urban street facility has two or more time-of-day signal timing plans, a separate study period should be established for each plan period.

The study period is the time interval (within a day) that is represented by the performance evaluation. It consists of one or more consecutive analysis periods. A typical study period is 1.0 to 6.0 h in duration and is stated to represent specific times of the day and days of the week (e.g., weekdays from 4:00 to 6:00 p.m.). If oversaturated conditions occur during the study period, at least the first analysis period should be undersaturated. The maximum study period duration is 24 h.

The geometric design elements and traffic control features of the facility must be unchanged during the study period. Thus, the intersection lane assignments and signal timing plan should be the same throughout the study period. In addition, if the directional distribution of traffic volume changes significantly during the day, separate study periods should be established for each time period where the directional distribution is relatively constant.

Reliability Reporting Period

The reliability reporting period represents the specific days over which the travel time distribution is to be computed. A typical reporting period for a reliability evaluation is 6 to 12 months. The period is specified by start and end dates as well as by the days of week being considered. The reliability reporting period is used with the study period to describe the temporal representation of the performance measure fully (e.g., average travel time on weekdays from 4:00 to 6:00 p.m. for the current year).

Demand Pattern Data

Demand pattern data are used by the reliability method to adjust the demand volumes in the base and alternative datasets to reflect demands during all the other time periods in the reliability reporting period. The data include (*a*) adjustment factors to account for demand variation by hour of day, day of week, and month of year and (*b*) adjustment factors to account for change in traffic demand due to weather conditions.

Time Period Adjustment Factors

The methodology requires day-of-week and month-of-year factors, expressed as ratios of the average day-of-week and average month-of-year demand. Also required are hour-of-day factors expressed as a percentage of annual average daily traffic (AADT). The specific factors needed are described as follows:

- Hour-of-day factors for each hour of the study period (up to 24, but typically six or fewer in practice),
- Day-of-week factors for each day included as part of the reliability reporting period (up to seven), and
- Month-of-year factors for each month included as part of the reliability reporting period (up to 12).

Default hour-of-day, day-of-week, and month-of-year traffic demand adjustment factors are given in Exhibit 17-5 through Exhibit 17-7, respectively. The factors should be replaced with data from permanent traffic count stations whenever available for streets that are similar to the subject facility and located near it.

Exhibit 17-5
Default Hour-of-Day Demand Ratios (ADT/AADT)

Hour Starting	Expressway Weekday	Expressway Weekend	Principal Arterial Weekday	Principal Arterial Weekend	Minor Arterial Weekday	Minor Arterial Weekend
Midnight	0.010	0.023	0.010	0.023	0.010	0.028
1 a.m.	0.006	0.015	0.006	0.014	0.006	0.023
2 a.m.	0.004	0.008	0.005	0.010	0.004	0.021
3 a.m.	0.004	0.005	0.005	0.006	0.002	0.008
4 a.m.	0.007	0.005	0.009	0.006	0.002	0.005
5 a.m.	0.025	0.009	0.030	0.010	0.007	0.005
6 a.m.	0.058	0.016	0.054	0.017	0.023	0.011
7 a.m.	0.077	0.023	0.071	0.024	0.067	0.018
8 a.m.	0.053	0.036	0.058	0.035	0.066	0.030
9 a.m.	0.037	0.045	0.047	0.046	0.054	0.048
10 a.m.	0.037	0.057	0.046	0.056	0.051	0.054
11 a.m.	0.042	0.066	0.050	0.054	0.056	0.057
Noon	0.045	0.076	0.053	0.071	0.071	0.074
1 p.m.	0.045	0.073	0.054	0.071	0.066	0.071
2 p.m.	0.057	0.074	0.063	0.072	0.060	0.069
3 p.m.	0.073	0.075	0.069	0.073	0.062	0.067
4 p.m.	0.087	0.075	0.072	0.073	0.063	0.071
5 p.m.	0.090	0.071	0.077	0.073	0.075	0.068
6 p.m.	0.068	0.063	0.062	0.063	0.070	0.067
7 p.m.	0.049	0.051	0.044	0.052	0.053	0.056
8 p.m.	0.040	0.043	0.035	0.044	0.044	0.049
9 p.m.	0.037	0.037	0.033	0.038	0.035	0.040
10 p.m.	0.029	0.032	0.026	0.033	0.033	0.035
11 p.m.	0.019	0.023	0.021	0.026	0.019	0.024

Source: Hallenbeck et al. (*9*).

Exhibit 17-6
Default Day-of-Week Demand Ratios (ADT/AADT)

Day	Demand Ratio
Sunday	0.87
Monday	0.98
Tuesday	0.98
Wednesday	1.00
Thursday	1.03
Friday	1.15
Saturday	0.99

Source: Hallenbeck et al. (*9*).

Exhibit 17-7
Default Month-of-Year Demand Ratios (ADT/AADT)

Month	Expressway	Principal Arterial	Minor Arterial
January	0.802	0.831	0.881
February	0.874	1.021	0.944
March	0.936	1.030	1.016
April	0.958	0.987	0.844
May	1.026	1.012	1.025
June	1.068	1.050	1.060
July	1.107	0.991	1.150
August	1.142	1.054	1.110
September	1.088	1.091	1.081
October	1.069	0.952	1.036
November	0.962	0.992	0.989
December	0.933	0.938	0.903

Source: Hallenbeck et al. (*9*).

Demand Change Factors

The three "demand change factors" account for a change in traffic demand due to weather conditions. One factor describes demand change during dry weather (by definition it has a value of 1.0). A second factor describes demand change during a rain event. The third factor describes demand change for a snow event. During a step of the methodology, the demand volume is multiplied by the demand change factor corresponding to the weather associated with a given analysis period. A factor less than 1.0 corresponds to a reduction in demand during the event.

Research indicates that urban street traffic demand tends to drop 15% to 30% during snow events (*10*). These motorists likely altered the start time of their commute or stayed home to avoid the bad weather. In the absence of local data, a default value of 0.80 may be used for snow events.

The research is less clear on the effect of rain on traffic demand. The effect of rain may vary with the trip purpose and the annual frequency of rain events in the vicinity of the subject facility. A default factor value of 1.0 is recommended for rain events. These default values are summarized in Exhibit 17-8. The input data item in the last row of this exhibit is discussed in the next subsection.

Exhibit 17-8
Default Values for Weather Events

Input Data Item	Default Value
Demand change factor for dry weather	1.00
Demand change factor for rain event	1.00
Demand change factor for snow event	0.80
Pavement runoff duration for snow event	0.5 h

Weather Data

Weather Event Statistics

A reliability evaluation requires the weather data identified in the following list. These data represent averages by month of year for a recent 10-year period.

- Total normal precipitation (in.),
- Total normal snowfall (in.),
- Number of days with precipitation of 0.01 in. or more (days),
- Normal daily mean temperature (°F), and
- Precipitation rate (in./h).

Default values for the aforementioned statistics are available from the National Climatic Data Center (NCDC) for 284 locations in the United States (*11, 12*). These default values are numerous, so they are not shown here. They are listed in the STREETVAL computational engine available in the online HCM Volume 4.

Duration of Pavement Runoff

The duration of pavement runoff for a snow event is required. It is defined as the period of time after the snow stops falling that snowpack (or ice) covers the pavement. After this time period elapses, the pavement is exposed and drying begins. This time is likely a function of traffic volume, snow depth, and agency snow removal capabilities. An appropriate local value should be established for the subject facility if that is possible. If such a value is not available, the default value provided in the last row of Exhibit 17-8 can be used.

Incident Data

Crash Location Categories

Chapter 16, Urban Street Facilities, defines segments as including portions of their bounding intersections (i.e., segments extend from the upstream intersection stop bar to the downstream intersection stop bar). For the purposes of reliability analysis, this definition must be modified to categorize each crash in accordance with the location of its occurrence (i.e., on the segment or at the intersection). The categorization of crashes by location is determined by using the definitions given in *Highway Safety Manual* (HSM) Section A.2.3, found in Appendix A of HSM Volume 2 (*13*). The HSM states: "Intersection crashes include crashes that occur at an intersection (i.e., within the curb limits) and crashes that occur on the intersection legs and are intersection-related. All crashes that are not classified as intersection or intersection-related crashes are considered to be roadway segment crashes."

Base Segment and Intersection Crash Frequency

The methodology requires the base crash frequency for each segment and for each intersection along the subject facility. The base crash frequency is an estimate of the expected crash frequency for the segment or intersection when no work zones are present or special events occur. The estimate should include all severity levels, including property-damage-only (PDO) crashes. Crash frequency is provided in units of crashes per year, regardless of the duration of the reliability reporting period.

Crash Frequency Adjustment Factors for Work Zones and Special Events

Crash frequency adjustment factors must be supplied for each work zone or special event for which an alternative dataset is assembled. One crash frequency adjustment factor is supplied for each segment and one is supplied for each intersection. They are used (at the appropriate step of the reliability methodology) to estimate the expected crash frequency when a work zone or special event is present. The estimate is obtained when the appropriate factor is multiplied by the base crash frequency for the segment or intersection. The result represents the expected crash frequency in a segment or at an intersection if the work zone or special event were present for 1 year.

The factor value should include consideration of the effect of the work zone or special event on traffic volume and crash risk. Volume may be reduced because of diversion, while changes in the roadway geometry and signal operation for a work zone or special event may increase the potential for a crash. To illustrate this concept, consider a work zone that is envisioned to increase crash risk by 100% (i.e., crash risk is doubled) and to decrease traffic volume by 50% (i.e., volume is halved). In this situation, the crash frequency adjustment factor is 1.0 (= 2.0 × 0.5). The analyst's experience with similar types of work zones or special events should be used to determine the appropriate adjustment factor value for the subject facility.

Crash Frequency Adjustment Factors for Inclement Weather

Inclement weather conditions can increase the likelihood of crashes. Crash frequency adjustment factors are required for the following conditions:

- Rainfall,
- Snowfall,
- Wet pavement (not raining), and
- Snow or ice on pavement (not snowing).

The crash frequency adjustment factor is the ratio of hourly crash frequency during the weather event to the hourly crash rate during clear, dry hours. It is computed by using one or more years of historical weather data and crash data for the region in which the subject facility is located.

The adjustment factor for a specific weather condition is computed from (*a*) the number of hours for which the weather condition exists for the year and (*b*) the count of crashes during those hours. An hourly crash frequency for the weather condition $f_{c,wea}$ is computed by dividing the crash count by the number of hours. By a similar technique, the hourly crash frequency is computed for dry pavement hours $f_{c,dry}$. The crash frequency adjustment factor for the weather condition $CFAF_{wea}$ is computed as the ratio of the two frequencies (i.e., $CFAF_{wea} = f_{c,wea}/f_{c,dry}$).

The crash frequency adjustment factor includes consideration of the effect of the weather event on traffic volume (i.e., volume may be reduced because of bad weather) and on crash risk (i.e., wet pavement may increase the potential for a crash). For example, if rainfall is anticipated to increase crash risk by 200% and to decrease traffic volume by 10%, the crash frequency adjustment factor for rainfall is 2.70 (= 3.0 × 0.9).

Exhibit 17-9 provides default values for the crash frequency adjustment factor for inclement weather. The other input data elements listed in the exhibit are discussed in the next subsection.

Exhibit 17-9
Default Values for Incidents

Input Data Element	Default Values
Crash frequency adjustment factor for weather conditions	Rainfall: 2.0 Wet pavement (not raining): 3.0 Snowfall: 1.5 Snow or ice on pavement (not snowing): 2.75
Incident detection time	2.0 min (all weather conditions)
Incident response time	Clear, dry: 15.0 min Rainfall: 15.0 min Wet pavement (not raining): 15.0 min Snowfall: 20.4 min Snow or ice on pavement (not snowing): 20.4 min

Source: Zegeer et al. (*1*).

Factors Influencing Incident Duration

The duration of an incident depends on a number of factors, including time to detect an incident, time to respond, and time to clear the incident. The incident detection time is the time period starting with the occurrence of the incident and ending when the response officials are notified of the incident. Incident response time is the time period from the receipt of incident notification by officials to the time the first response vehicle arrives at the scene of the incident. This time will likely vary among jurisdictions and facilities, depending on the priority placed on street system management and the connectivity of the street system. Incident clearance time is the time from the arrival of the first response vehicle to the time when the incident and service vehicles no longer directly affect travel on the roadway. This time varies by incident location, type, and severity.

Response and clearance times are weather-dependent; clearance times are also dependent on the incident severity and location (e.g., shoulder versus travel lanes). The following values are required:

- Incident detection time, in minutes;
- Incident response times, in minutes, for five weather categories (dry, rainfall, snowfall, wet pavement, snow or ice on pavement); and
- Incident clearance times, in minutes, by street location (segment or intersection), incident type (crash or noncrash), lane location (shoulder, one lane, two or more lanes), severity (fatal/injury or PDO), and weather condition (dry, rainfall, wet pavement, snowfall or snow or ice on pavement) (96 total values).

Default values for incident detection time and incident response time are provided in Exhibit 17-9. Default values for incident clearance time are provided in Exhibit 17-10. The default distributions for segments and intersections are the same in this exhibit. Segments and intersections are differentiated because the method allows the analyst to provide different clearance times for segments and intersections when local values are available.

Exhibit 17-10
Default Incident Clearance Times

Street Location	Event Type	Lane Location	Severity[a]	Clearance Time by Weather Condition (min) Dry	Rain-fall	Wet Pavement	Snow or Ice[b]
Segment	Crash	One lane	FI	56.4	42.1	43.5	76.7
			PDO	39.5	28.6	29.7	53.7
		2+ lanes	FI	56.4	42.1	43.5	76.7
			PDO	39.5	28.6	29.7	53.7
		Shoulder	FI	56.4	42.1	43.5	76.7
			PDO	39.5	28.6	29.7	53.7
	Non-crash	One lane	Breakdown	10.8	5.6	5.7	14.7
			Other	6.7	2.4	2.8	9.1
		2+ lanes	Breakdown	10.8	5.6	5.7	14.7
			Other	6.7	2.4	2.8	9.1
		Shoulder	Breakdown	10.8	5.6	5.7	14.7
			Other	6.7	2.4	2.8	9.1
Signalized intersection	Crash	One lane	FI	56.4	42.1	43.5	76.7
			PDO	39.5	28.6	29.7	53.7
		2+ lanes	FI	56.4	42.1	43.5	76.7
			PDO	39.5	28.6	29.7	53.7
		Shoulder	FI	56.4	42.1	43.5	76.7
			PDO	39.5	28.6	29.7	53.7
	Non-crash	One lane	Breakdown	10.8	5.6	5.7	14.7
			Other	6.7	2.4	2.8	9.1
		2+ lanes	Breakdown	10.8	5.6	5.7	14.7
			Other	6.7	2.4	2.8	9.1
		Shoulder	Breakdown	10.8	5.6	5.7	14.7
			Other	6.7	2.4	2.8	9.1

Source: Zegeer et al. (*1*).
Notes: [a] FI = fatal or injury crash; PDO = property-damage-only crash.
[b] Applies to snowfall and to snow or ice on pavement (but not snowing).

In general, an analyst should supply local values for the incident duration factors when the reliability analysis is testing the effects of traffic management measures that influence incident detection, response, or clearance.

Incident Location Distribution

The incident location distribution is used by the incident generation procedure to assign incidents to specific locations on the facility. Research indicates that this distribution varies by incident location, type, and severity. The following incident proportions are required:

- Proportion of crash and noncrash incidents by street location (segment or intersection) (four total values); proportions should total 1.000 for a given street location;
- Proportion of shoulder, one-lane, and two-or-more-lane incidents by street location and event type (crash or noncrash) (12 total values); proportions should total 1.000 for a given street location and event type combination; a 0.000 proportion should be assigned to values involving a shoulder location if no shoulders exist on the facility;
- Proportion of fatal or injury and PDO crashes by street location and lane location (12 total values); proportions should total 1.000 for a given street location and lane location combination; and
- Proportion of breakdown and other noncrash incidents by street location and lane location (12 total values); proportions should total 1.000 for a given street location and lane location combination.

The four proportions identified in the previous list are multiplied together to obtain the desired incident location distribution factors. One factor is obtained for each combination of street location, incident type, incident location, and incident severity. The computed factors should total 1.000 for a given street location.

Default values for these factors are provided in the last column of Exhibit 17-11 and Exhibit 17-12. The default distribution of incident lane location is based on facilities with outside shoulders. The distribution is modified accordingly when shoulders are not present on the subject facility. The first exhibit provides the distribution for urban streets with shoulders. The second exhibit provides the distribution for urban streets without shoulders.

Exhibit 17-11
Default Incident Distribution with Shoulder Presence

Street Location	Incident Type: Type	Incident Type: Proportion	Incident Location: Lanes Affected	Incident Location: Proportion	Incident Severity: Severity[a]	Incident Severity: Proportion	Joint Proportion
Segment	Crash	0.358	1 lane	0.335	FI	0.304	0.036
					PDO	0.696	0.083
			2+ lanes	0.163	FI	0.478	0.028
					PDO	0.522	0.030
			Shoulder	0.502	FI	0.111	0.020
					PDO	0.889	0.160
	Non-crash	0.642	1 lane	0.849	Breakdown	0.836	0.456
					Other	0.164	0.089
			2+ lanes	0.119	Breakdown	0.773	0.059
					Other	0.227	0.017
			Shoulder	0.032	Breakdown	0.667	0.014
					Other	0.333	0.007
						Total:	1.000
Signalized intersection	Crash	0.310	1 lane	0.314	FI	0.378	0.037
					PDO	0.622	0.061
			2+ lanes	0.144	FI	0.412	0.018
					PDO	0.588	0.026
			Shoulder	0.542	FI	0.109	0.018
					PDO	0.891	0.150
	Non-crash	0.690	1 lane	0.829	Breakdown	0.849	0.486
					Other	0.151	0.086
			2+ lanes	0.141	Breakdown	0.865	0.084
					Other	0.135	0.013
			Shoulder	0.030	Breakdown	0.875	0.018
					Other	0.125	0.003
						Total:	1.000

Source: Zegeer et al. (*1*).
Note: [a] FI = fatal or injury crash; PDO = property-damage-only crash; other = not breakdown (e.g., debris).

Exhibit 17-12
Default Incident Distribution Without Shoulder Presence

Street Location	Incident Type: Type	Incident Type: Proportion	Incident Location: Lanes Affected	Incident Location: Proportion	Incident Severity: Severity[a]	Incident Severity: Proportion	Joint Proportion
Segment	Crash	0.358	1 lane	0.837	FI	0.304	0.091
					PDO	0.696	0.209
			2+ lanes	0.163	FI	0.478	0.028
					PDO	0.522	0.030
	Non-crash	0.642	1 lane	0.881	Breakdown	0.836	0.473
					Other	0.164	0.093
			2+ lanes	0.119	Breakdown	0.773	0.059
					Other	0.227	0.017
						Total:	1.000
Signalized intersection	Crash	0.310	1 lane	0.856	FI	0.378	0.100
					PDO	0.622	0.165
			2+ lanes	0.144	FI	0.412	0.018
					PDO	0.588	0.026
	Non-crash	0.690	1 lane	0.859	Breakdown	0.849	0.503
					Other	0.151	0.089
			2+ lanes	0.141	Breakdown	0.865	0.084
					Other	0.135	0.013
						Total:	1.000

Source: Zegeer et al. (*1*).
Note: [a] FI = fatal or injury crash; PDO = property-damage-only crash; other = not breakdown (e.g., debris).

Work Zone and Special Event Data

Work zones and special events require the use of alternative datasets that specify the demand, geometric, and traffic control conditions in effect during the work zone or special event. A schedule (i.e., start and end dates) is also required that specifies when the work zone is in effect or when the special event takes place.

Traffic Counts

The date and time of the traffic count represented in the base dataset are required inputs. If the base dataset demands are computed by using planning procedures, they are assumed to represent average day volumes. In this case, a date does not need to be provided by the analyst. However, the time of day for which the estimated volumes apply is still needed. The date and time of the traffic count represented in an alternative dataset are also required inputs.

Data Sources

Reliability (as measured by TTI or PTI) can vary widely with the characteristics of a particular facility. Therefore, analysts are encouraged to use local values representative of local demand, weather, and incident patterns when the data are available. In addition, analysts must supply local values for work zones and special events if they wish to account for these effects in a reliability analysis. This subsection identifies potential sources of these data.

Demand Pattern Data

The best potential source of demand pattern data is a permanent traffic recorder (PTR) located along the facility. Alternatively, an analyst may be able to use data from a PTR located along a similar facility in the same geographic area. Many state departments of transportation produce compilations of data from their PTRs and provide demand adjustment factors by time of day, day of week, and month of year by facility and area type. The analyst is reminded that measured volumes are not necessarily reflective of demands. Upstream bottlenecks may limit the volume reaching a PTR or other observation point.

Weather Data

NCDC provides rainfall, snow, and temperature statistics for thousands of locations through its website (*11*) and average precipitation rate data in the *Rainfall Frequency Atlas* (*12*).

A weather station that a transportation agency has installed along the study facility may also be able to provide the required data, if the agency stores and archives the data collected by the station. A 10-year weather dataset is desirable for capturing weather events that are rare but have a high impact.

Finally, analysts should consider the location of the facility relative to the weather station. Elevation differences, proximity to large bodies of water, and other factors that create microclimates may result in significant differences in the probabilities of certain types of weather events (e.g., snow, fog) on the facility and at the weather station.

Incident Data

The base crash frequency for a segment or intersection can be computed with the predictive method in Chapter 12 of the 2010 HSM (*13*). If this method cannot be used, the base crash frequency of a segment or an intersection can be estimated on the basis of its 3-year crash history. However, crashes that occur when work zones and special events are present should be removed from the crash data. In this situation, the expected crash frequency is computed as the count of crashes during times when work zones and special events are not present divided by the time period when work zones and special events are not present. Thus, if 15 crashes were reported during a recent 3-year period and five of the crashes occurred during a 6-month period when a work zone was present, the base crash frequency is estimated as 4.0 crashes per year [= (15 – 5)/(3 – 0.5)]. A technique for distinguishing between segment- and intersection-related crashes is described in Appendix A of Part C of the 2010 HSM (*13*).

Work Zone Data

A schedule of long-term work zones should be obtained from the roadway operating agency. The schedule should indicate the days and times when the work zone will be in force and the portions of the roadway that will be affected.

Work zones that vary in intensity (e.g., one lane closed on some days and two lanes closed on others) or that affect different segments at different times will need to be provided as separate alternative datasets.

When detailed traffic control plans for each work zone are available, they should be consulted to determine the starting and ending locations of lane closures, along with any reductions in the posted speed. When detailed plans are not available, the agency's standard practices for work zone traffic control can be consulted to determine the likely traffic control that would be implemented, given the project's characteristics.

Special Event Data

Special events are short-term events, such as major sporting events, concerts, and festivals, that produce intense traffic demands on a facility for limited periods. Special traffic control procedures may need to be implemented to accommodate the traffic demands before, during, or after these events.

The analyst should identify whether any events that occur in or near the study area warrant special treatment. If so, a schedule for the event (date, starting time, duration) should be obtained. Some types of events also have varying intensities that will require separate treatment (e.g., a sold-out baseball game compared with a lower-attendance midweek game). Recurring events may have developed special traffic control procedures; if so, these plans should be consulted to identify any changes required from base conditions. Alternative datasets will be needed for each combination of special event venue and event intensity.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the reliability of an urban street facility. The methodology is computationally intense and requires software to implement. The intensity stems from the need to create and process the input and output data associated with the hundreds or thousands of scenarios considered for a typical reliability reporting period.

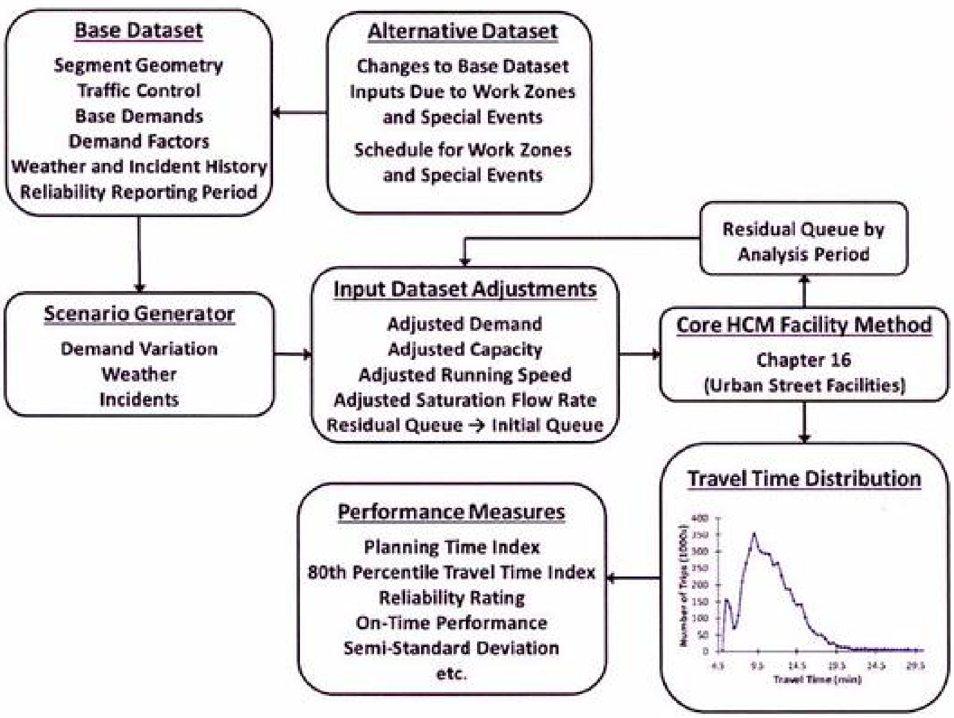
The objective of this section is to introduce the analyst to the calculation process and discuss the key analytic procedures. Important equations, concepts, and interpretations are highlighted.

The computational details of the methodology are provided in Chapter 29, Urban Street Facilities: Supplemental. The STREETVAL computational engine provided in the Technical Reference Library in the online HCM Volume 4 represents the most detailed description of the methodology.

Framework

The sequence of calculations in the reliability methodology is shown in Exhibit 17-13. There are five main steps: (*a*) establishing base and alternative datasets, (*b*) generating scenarios, (*c*) evaluating each scenario with the Chapter 16 methodology, (*d*) compiling travel times for each analysis period in the reliability reporting period, and (*e*) producing reliability performance measures.

Exhibit 17-13
Reliability Methodology Framework



Data Depository

Every reliability analysis requires a base dataset. This dataset describes the traffic demand, geometry, and signal timing conditions for the intersections and segments along the facility during the study period when no work zones are present and no special events occur.

Additional datasets are used, as needed, to describe the conditions when a specific work zone is present or when a special event occurs. These datasets are called the alternative datasets. One alternative dataset is used for each time period during the reliability reporting period when a specific work zone is present, a specific special event occurs, or a unique combination of these conditions occurs during the study period.

As a first step in the reliability evaluation, the analyst develops the aforementioned datasets. Then the analyst assembles the input data needed for the reliability methodology. These input data are described in the previous subsection titled Required Data and Sources.

Scenario Generation

The scenario generation stage consists of four sequential procedures: (*a*) weather event generation, (*b*) traffic demand variation generation, (*c*) traffic incident generation, and (*d*) scenario dataset generation. Each procedure generates in chronological order the set of analysis periods that make up the reliability reporting period. This subsection gives an overview of the scenario generation process; a detailed description is provided in Chapter 29, Urban Street Facilities: Supplemental.

Weather Event Generation

Future research may indicate that additional weather types affect arterial operation. At this time, available research supports assessment of rain and snow events on arterial operation.

The weather event procedure generates rain and snow events during the reliability reporting period. The dates, times, types (i.e., rain or snow), and durations of severe weather events are generated. These data are used to adjust the saturation flow rate and speed of facility traffic for each analysis period. The procedure also predicts the time after each weather event that the pavement remains wet or covered by snow or ice, since the presence of these conditions influences running speed and intersection saturation flow rate.

Traffic Demand Variation Generation

The traffic demand variation procedure identifies the appropriate traffic demand adjustment factors for each analysis period in the reliability reporting period. A set of factors accounts for systematic demand variation by hour of day, day of week, and month of year.

Traffic Incident Generation

The traffic incident procedure generates incident dates, times, and durations. It also determines incident types (i.e., crash or noncrash), severity levels, and locations on the facility. Location is defined by the intersection or segment on which the incident occurs and whether the incident occurs on the shoulder, in one lane, or in multiple lanes. The procedure incorporates weather and traffic demand variation information from the previous procedures in generating incidents.

Scenario Dataset Generation

The scenario dataset generation procedure uses the results from the preceding procedures to develop one HCM dataset for each analysis period in the reliability reporting period. Each analysis period is considered to be one scenario. The base dataset is modified to reflect conditions present during a given analysis period. Traffic volumes are modified at each intersection and driveway. Saturation flow rates are adjusted at intersections influenced by an incident or a weather event, and speeds are adjusted for segments influenced by an incident or a weather event. Dates and times represent a common basis for tracking events and conditions from one analysis period to the next.

Facility Evaluation

The facility evaluation stage consists of two tasks that are repeated in sequence for each analysis period. The analysis periods are evaluated in chronological order.

First, the dataset associated with a given analysis period is evaluated by using the urban street facility methodology. The performance measures output by the methodology are then archived.

Second, the dataset associated with the next analysis period is modified, if necessary, on the basis of the results of the current analysis period. Specifically, the initial queue input value for the next analysis period is set equal to the residual queue output for the current analysis period.

Performance Summary

The performance summary stage consists of two sequential tasks. First, the analyst identifies a specific direction of travel and the performance measures of interest. The desired performance measures are extracted from the facility evaluation archive for each analysis period in the reliability reporting period. Available measures, as defined in Chapter 18, Urban Street Segments, are as follows:

- Travel time,
- Travel speed,
- Stop rate,
- Running time, and
- Through delay.

The analyst also indicates whether the performance measures of interest should be representative of the entire facility or a specific segment. The first three measures in the above list are available for facility evaluation. All five measures are available for segment evaluation. At the conclusion of this task, the collected data represent observations of the performance measures for each analysis period occurring during the reliability reporting period (or a sampled subset thereof).

Next, the selected performance measure data are summarized by using the following statistics:

- Average;
- Standard deviation;
- Skewness;
- Median;
- 10th, 80th, 85th, and 95th percentiles; and
- Number of observations.

In addition, the "average" base free-flow speed is always reported. This measure is computed as the arithmetic mean of the base free-flow speed for each scenario in the reliability reporting period. It can be used with one or more of the distribution statistics to compute various variability and reliability measures, such as the TTI and the reliability rating.

Interpretation of Results

Identifying Reliability Problems

In a perfect world, all urban street facilities would be perfectly reliable. They would have mean TTIs and PTIs of 1.00 or better. Since operating a perfectly reliable facility is not a realistic standard, an agency must distinguish between less than perfect—but still acceptable—reliability and unacceptable reliability. This is obviously a choice that each agency must make. This subsection provides guidance on the factors and criteria that a transportation agency may wish to consider in making its selection, but the final decision is up to the agency.

Criterion No. 1: How Does Reliability Compare with Agency Congestion Management Policy?

An agency may have a policy of delivering a certain minimum speed or maximum travel time on its urban streets, or a maximum acceptable delay per signal or per mile. If so, the computation of the reliability statistics can be modified on the basis of this information so that failures to meet agency policy can be identified more quickly. This approach is illustrated in the following paragraphs.

Minimum speed policy. If the agency has a minimum acceptable facility speed policy, this information can be used to compute the reliability statistics that use the acceptable speed as a baseline (instead of the base free-flow speed). Determining the extent to which the facility meets the agency's target performance level by comparing the computed reliability statistic with the target value of 1.00 is then relatively easy. The result of using the policy speed instead of the base free-flow speed is to neglect travel time reliability when speeds exceed the agency's minimum acceptable threshold.

For example, if the agency's congestion management policy is to deliver speeds in excess of 40 mi/h, the policy travel times are computed by using the facility length divided by 40 mi/h and converting the result to minutes. The policy travel time index is then computed with the following equation:

Equation 17-1

$$TTI_{\text{policy}} = \frac{\overline{TT}_F}{\overline{TT}_p}$$

with

$$\overline{TT}_F = \frac{1}{N_d\,N_{ap}} \times \frac{3{,}600}{5{,}280} \times \sum_{d=1}^{N_d} \sum_{ap=1}^{N_{ap}} \sum_{i=1}^{m} \frac{L_i}{S_{T,seg,i,ap,d}}$$

$$\overline{TT}_p = \frac{3{,}600}{5{,}280} \times \sum_{i=1}^{m} \frac{L_i}{40}$$

where

TTI_{policy} = policy travel time index, based on the agency's policy (or target) travel time for the facility (unitless);

$\overline{TT}_F$ = average travel time for through trips on the facility during the reliability reporting period (s);

$\overline{TT}_p$ = agency's maximum acceptable travel time for through trips on the facility during the reliability reporting period (s);

L_i = length of segment i (ft);

m = number of segments on the facility;

N_{ap} = number of analysis periods in 1 day (i.e., study period);

N_d = number of days in the reliability reporting period; and

$S_{T,seg,i,ap,d}$ = travel speed of through vehicles for segment i during analysis period ap and day d (mi/h).

Values of 1.00 or less for TTI_{policy} mean that the agency's congestion management policy is being met on average over the course of the reliability reporting period. Values greater than 1.00 mean that the facility is failing to meet the agency's policy on average.

Maximum acceptable delay. If the agency has a maximum acceptable delay standard per mile or per signal, the mean TTI can be readily converted into equivalent delay estimates for the facility and compared with the agency standard. The following equations illustrate this conversion.

Equation 17-2

$$d_{\text{trip}} = \overline{TT}_{fo,F} \times (TTI_{\text{mean}} - 1)$$

$$d_{\text{mile}} = \frac{\overline{TT}_{fo,F}}{\sum_{i=1}^{m} L_i} \times (TTI_{\text{mean}} - 1)$$

$$d_{\text{signal}} = \frac{\overline{TT}_{fo,F}}{N_s} \times (TTI_{\text{mean}} - 1)$$

with

$$\overline{TT}_{fo,F} = \frac{1}{N_d\, N_{ap}} \times \frac{3{,}600}{5{,}280} \times \sum_{d=1}^{N_d} \sum_{ap=1}^{N_{ap}} \sum_{i=1}^{m} \frac{L_i}{S_{fo,seg,i,ap,d}}$$

where

d_{trip} = average delay per trip (s/veh);

d_{mile} = average delay per mile (s/veh);

d_{signal} = average delay per signal (s/veh);

TTI_{mean} = mean travel time index (unitless);

$\overline{TT}_{fo,F}$ = average travel time for through trips at the base free-flow speed on the facility during the reliability reporting period (s);

$S_{fo,seg,i,ap,d}$ = base free-flow speed of through vehicles for segment i during analysis period ap and day d (mi/h); and

N_s = number of signals within study section of facility (unitless).

These delay values can also be computed by using the PTI (or any other percentile TTI value) by substituting the desired TTI value for TTI_{mean} in the appropriate equation. A policy TTI value can also be substituted.

Criterion No. 2: How Does Reliability Compare with Other Facilities?

To address this question, the agency ranks the reliability results for a given facility against those of other facilities it operates and prioritizes improvements to its facilities with the worst reliability accordingly. Of course, this approach requires that the agency collect reliability data for its facilities so that the agency's facility investments can be properly ranked according to need.

Criterion No. 3: How Does Reliability Compare with HCM LOS?

This criterion involves translating reliability results into more traditional HCM LOS results with which decision makers may be more comfortable. The reliability results are used to identify what percentage of time a facility is operating at an unacceptable LOS and in determining a percentage of time that is unacceptable.

For example, the agency's standard may be LOS D. The reliability results may show that the facility operates at LOS E or worse during 5% of the weekday peak periods over the course of a year. This may be an acceptable risk for the agency if the costs of improvements to eliminate the 5% risk are high.

The inverse of the PTI represents the 95th percentile slowest through-trip speed on the facility divided by the base free-flow speed. Thus, the inverse PTI can be used to determine whether the facility will operate at a LOS acceptable to the agency at least 95% of the time. In this regard, the inverse PTI (multiplied by 100 to yield a percentage) is compared with the base-free-flow-speed percentage associated with the LOS considered acceptable. The percentage associated with each LOS is described in the subsection titled LOS Criteria, Motorized Vehicle Mode, in Section 2 of Chapter 16.

Diagnosing the Causes of Reliability Problems

Exhibit 17-14 identifies seven sources of congestion and unreliability and shows how they interact with each other. The starting point in traditional analysis is to take a fixed capacity and a fixed volume to develop an estimate of delay, usually for "typical" conditions. However, in the field, both physical capacity and demand vary because of roadway disruptions, travel patterns, and traffic control devices. These conditions not only decrease available capacity or cause volatility in demand but also interact with each other. For example, both inclement weather and work zones can lead to an increase in incidents.

Thus, diagnosing the relative contribution of different causes of unreliability involves identifying the causes individually and in combination. Depending on the purpose of the evaluation, various approaches may be taken for assigning the proportional responsibility to individual causes when two or more are acting in combination.

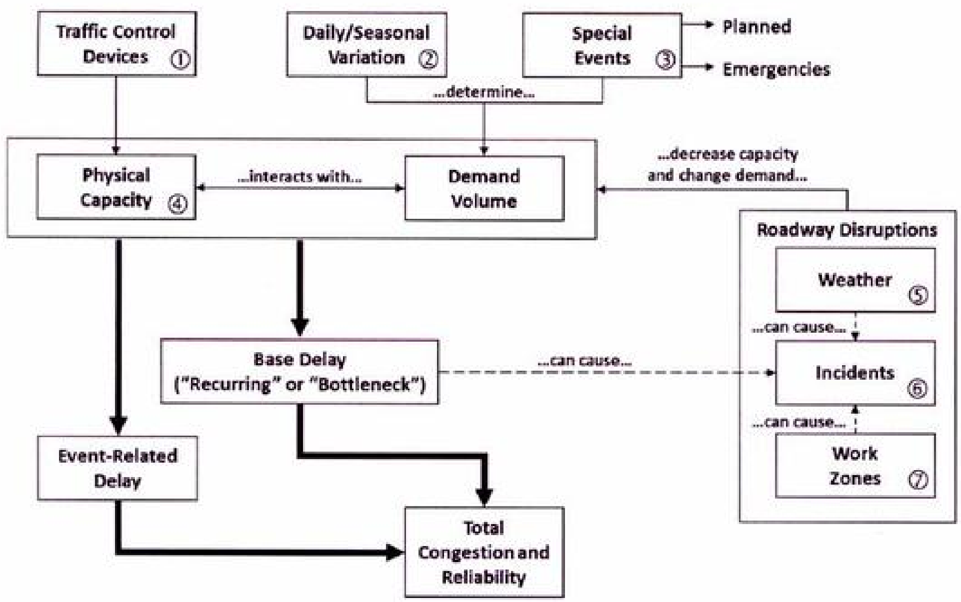


Exhibit 17-14
Interrelationship Between Causes of Congestion and the Facility

Selecting a Performance Measure

To identify the relative effects of different causes on the travel time reliability of the facility, computation of total vehicle (or person) hours of delay summed over the entire reliability reporting period is recommended. This measure of effectiveness takes into account both the severity of the event (i.e., demand surge, incident, weather) and its frequency of occurrence within the reliability reporting period. Exceptionally severe but rare events may add relatively little to the total annual delay experienced by the facility. Moderate but frequent events will often have a greater effect on total annual delay.

Generating a Simplified Matrix of Causes

Identifying patterns of results in several thousand scenarios is impractical, so consolidation of the many scenarios into a matrix of congestion causes along the lines of Exhibit 17-15 is recommended. This is best done by combining similar scenarios that individually contribute less than 1% to annual delay. In the example shown in Exhibit 17-15, the numerous severe weather events (rain, snow, etc.) have been consolidated into a single "bad weather" category because severe weather is relatively infrequent at this site. The results from the original analysis of multiple demand levels have similarly been consolidated into three levels (low, medium, high).

Exhibit 17-15
Example Matrix Allocating Annual Vehicle Hours of Delay by Cause

Incidents	Low Demand Fair Weather	Low Demand Bad Weather	Moderate Demand Fair Weather	Moderate Demand Bad Weather	High Demand Fair Weather	High Demand Bad Weather	Total
None	596 (2%)	407 (1%)	818 (3%)	362 (1%)	6,240 (23%)	956 (4%)	9,379 (34%)
1 lane closed	2,363 (9%)	92 (<1%)	2,097 (8%)	61 (<1%)	9,102 (33%)	119 (<1%)	13,834 (51%)
2 lanes closed	194 (1%)	13 (<1%)	189 (1%)	9 (<1%)	907 (3%)	17 (<1%)	1,329 (5%)
3 lanes closed	621 (2%)	40 (<1%)	468 (2%)	23 (<1%)	1,510 (6%)	32 (<1%)	2,694 (10%)
Total	3,774 (14%)	552 (2%)	3,572 (13%)	455 (2%)	17,759 (65%)	1,124 (4%)	27,236 (100%)

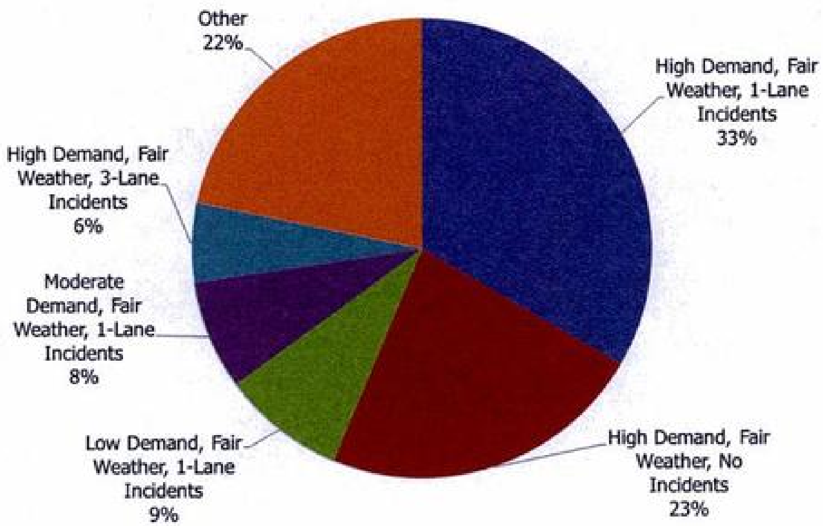
Diagnosing Primary Causes of Unreliability

The diagnosis proceeds by first examining the cells of the matrix to identify those with the largest annual delay values. For example, examination of the cells in Exhibit 17-15 yields the following conclusions:

- The single greatest cause of annual delay on the example facility is incidents closing a single lane under high-demand conditions on fair-weather days. They account for 33% of the annual delay on the facility.
- The next largest occurrence of annual delay happens under high-demand, fair-weather, no-incident conditions. They account for 23% of the annual delay on the facility.
- The third and fourth largest annual delays occur when incidents close a single lane under fair-weather conditions with low- to moderate-demand conditions. Together, these scenarios account for 17% of the annual delay on the facility.
- The fifth largest annual delays are accumulated when incidents close three lanes under high-demand and fair-weather conditions.

Exhibit 17-16 shows that the top five cells in Exhibit 17-15 account for about 78% of the annual delay on the facility.

Exhibit 17-16
Example Pie Chart of Congestion Causes



The next step is to examine the row and column totals to determine whether a single cause stands out. For example, examination of the row and column totals in Exhibit 17-15 yields the following conclusions:

- The highest row or column total annual delay occurs in high-demand, fair-weather conditions. Recurring congestion is therefore a significant source of delay on this example facility. High-demand conditions account for 65% of the annual delay on the facility.
- The next highest row or column total occurs when incidents close one lane on the facility. Incidents blocking a single lane account for 51% of the delay on the facility.
- Bad weather is a minor cause of annual delay on the facility.

Developing a Treatment Plan

The conclusions from the example shown in Exhibit 17-15 suggest the following options that are likely to have the greatest effect on improving reliability in the example facility:

- Measures to reduce high-demand conditions or to increase capacity to address recurring congestion, and
- Measures to manage incidents that close a single lane.

The diagnostic process also indicates that in this example, bad weather and extreme incidents (closures of two or more lanes), despite their severity when they happen, are minor contributors to total annual delay on the facility.

The particular example used here was from a state with relatively mild weather. The results would likely be different on facilities in other parts of the country.

4. EXTENSIONS TO THE METHODOLOGY

ACTIVE TRAFFIC AND DEMAND MANAGEMENT STRATEGIES

ATDM strategies are essentially real-time changes in geometry, lane assignment, or traffic control that are implemented in the time frame of a few seconds. However, macroscopic deterministic models (like the methods described in the HCM) are generally limited to the evaluation of strategies that have a fixed geometry, lane assignment, and traffic control plan during the analysis time frame of 15 min to 1 h.

On the basis of these observations, the HCM appears unable to evaluate an ATDM strategy reliably, and to a certain extent that is true. However, in making investment decisions, the focus is not just on the next few seconds. Instead, the focus is on how the investment will perform over the coming years. For this kind of ATDM strategy evaluation, macroscopic deterministic models (like the methods in the HCM) are appropriate tools. The key is to identify where and how these tools might be adapted to recognize the superior efficiency of ATDM strategies (by delivering improved productivity) when compared with more static management approaches.

At this point relatively little research has been conducted on the demand, capacity, speed, and delay effects of urban street ATDM strategies. Therefore, the guidance in this section is general; it leaves some decisions up to the discretion of the analyst. The focus is on using the methodologies in HCM Chapters 16 to 23 to evaluate ATDM strategies for the purpose of making investment decisions.

EXTENSIONS FOR SPECIFIC TACTICS

Arterial Monitoring Tactics

Arterial monitoring tactics consist of an improved ability to detect vehicles and pedestrians at a finer level of detail than is possible with conventional pavement loop detectors. In the future, connected vehicles and pedestrians (using cell phones) may be able to place calls to the signal system in advance of arrival and provide sufficient information to enable the system to sort out the relative priorities of the various vehicles and pedestrians and assign signal times accordingly. Special guidance may be provided to visually impaired pedestrians arriving at a signalized intersection.

All of the aforementioned monitoring tactics provide additional (and more accurate) information to the signal system controller. However, the impact of a specific measure on urban street performance will depend on what the control system does with the additional information. The ability to evaluate more accurate monitoring will generally require no modification or extension of the HCM procedures because they are based on the assumption that the controller is fully aware of when vehicles and pedestrians arrive at the intersection. One exception would be if the monitoring tactic's function is tied to changes in demand or the anticipated demand. In this case, the HCM procedures would need to be revised to replicate the control system's response to the changing demand.

Signal and Speed Control Tactics

Signal and speed control tactics include adaptive control priority service for emergency response vehicles, transit vehicles, freight vehicles, and pedestrians. The means by which each measure might be evaluated by using the HCM is described in the following paragraphs.

Adaptive Control

Adaptive control is accomplished through second-by-second optimization of signal timings according to the current monitor information and the priorities assigned to each vehicle and pedestrian type by the operating agency. To evaluate this measure by using the HCM, some extensions are needed for the following computational steps of the motorized vehicle methodology in Chapter 18, Urban Streets Segments:

- Step 2: Determine running time.
- Step 3: Determine proportion arriving during green.

In the case of adaptive signal control, the phase splits used with the HCM method should be large enough to accommodate all of the predicted average hourly flows, subject to any limits on maximum phase length incorporated into the adaptive control algorithm.

The intersection peak hour factor should be set to 1.00 and only average hourly flows used in the HCM analysis, since the adaptive control will adapt the effective cycle length to address any fluctuations in demand within the hour.

The free-flow speed and running times produced by Step 2, Chapter 18, will probably need to be adjusted to reflect the lower likelihood of stopping with adaptive control (for suitably favorable demand conditions).

Similarly, the proportion arriving during green, produced in Step 3, Chapter 18, will need to be adjusted to reflect the lower likelihood of arriving on red with adaptive control (for suitably favorable demand conditions).

Under high-demand conditions, an adaptive control algorithm may revert to essentially pretimed control, in which case the HCM methodology for pretimed control can be used without adaptation.

If a range of demand conditions are expected, a set of high-, medium-, and low-demand scenarios should be evaluated with the reliability methodology described in Section 3.

Priority of Emergency Vehicles, Transit Vehicles, Freight, and Pedestrians

The HCM methodology can be used to evaluate the effects of priority treatments by performing a reliability analysis that identifies different evaluation scenarios. One scenario is developed for each priority treatment condition (e.g., bus present so transit priority invoked, no bus so transit priority not invoked). The HCM methodology is then used to evaluate performance associated with each scenario. The results are weighted by the probability of each priority call to obtain annual performance effects.

The HCM methods for estimating free-flow speed, running time, and the proportion arriving on green should be modified in each scenario to reflect the conditions for each priority call.

Speed Harmonization

Speed harmonization involves dynamically slowing traffic in advance of queues, incidents, and lane closures and then directing traffic to the remaining lanes. This ATDM strategy will affect lane utilization, free-flow speed, running time, and the proportion of vehicles arriving on green in the HCM analyses. The appropriate speed harmonization scenarios should be created and then evaluated with the reliability methodology described Section 3.

Geometric Configuration Tactics

Geometric configuration tactics dynamically reassign traffic movements to different lanes in response to demand surges. For example, the right-hand through lane may become an exclusive right-turn lane during periods of high right-turn demand.

Dynamic lane assignments, reversible lanes, and dynamic turn lane restrictions are evaluated in the HCM by using scenarios. One scenario is created for each desired lane configuration. The reliability methodology described in Section 3 is then used to evaluate the performance of each scenario and combine the results into an overall assessment of the whole year performance of the tactic.

Demand Modification Tactics

Demand modification tactics seek to change the demand for the facility. Traveler information, route guidance, dynamic parking, and congestion pricing are evaluated in the HCM by using scenarios. The analyst must estimate the likely demand effects of the tactic and then input this demand into the HCM analysis. One or more scenarios are created for each desired demand modification tactic. The reliability methodology described in Section 3 is then used to evaluate the performance of each scenario and combine the results into an overall assessment of the whole-year performance of the tactic.

5. APPLICATIONS

EXAMPLE PROBLEMS

Chapter 29, Urban Street Facilities: Supplemental, describes the application of the reliability methodology through the use of example problems. There is one example problem illustrating the use of the methodology to diagnose causes of reliability problems and one example problem illustrating the use of the methodology for alternatives analysis.

ANALYSIS TECHNIQUES

This subsection describes techniques for effective use of the reliability methodology.

Work Zones and Special Events

Work zones and special events influence traffic demand levels and travel patterns. To minimize the impact of work zones and special events on traffic operation, agencies responsible for managing traffic in the vicinity of a work zone or special event often reallocate some traffic lanes or alter the signal operation to increase the capacity of specific traffic movements. These characteristics mean that the effect of each work zone and special event on facility performance is unique. Multiple work zones and special events can occur during the reliability reporting period.

The reliability methodology incorporates work zone and special event influences in the evaluation results. However, the analyst must describe each work zone and special event by using an alternative dataset. Each dataset describes the traffic demand, geometry, and signal timing conditions when the work zone is present or the special event is under way. A start date and a duration are associated with each dataset.

The presence of a work zone can significantly affect traffic demand levels. The extent of the effect will depend partly on the availability of alternative routes, the number of days the work zone is in operation, and the volume-to-capacity ratio of the segment or intersection approach within the work zone.

When the reliability methodology is used, the analyst must provide estimates of traffic demand volumes during the work zone or special event. The estimates should reflect the effect of diversion and can be based on field measurements, judgment, or areawide traffic planning models. They are recorded by the analyst in the corresponding alternative dataset.

The analyst must have information about lane closures, alternative lane assignments, and special signal timing that is present during the work zone or special event. The information can be based on agency policy or experience with previous work zones or events. The available lanes, lane assignments, and signal timing are recorded by the analyst in the corresponding alternative dataset.

Multiple Study Periods

The geometric design elements, traffic control features (including signal timing plans), and directional distribution of traffic are assumed to be constant during the study period. If any of these factors varies significantly during certain periods of the day (e.g., morning peak or evening peak), each unique period should be the focus of a separate reliability evaluation. In this regard, each unique period represents one study period.

When multiple study period evaluations are undertaken for a common facility, the set of analysis period averages for each evaluation can be merged to evaluate the overall reliability. In this manner, the combined data for a given performance measure represent the distribution of interest. The various reliability measures are then quantified by using this combined distribution.

Alternatives Analysis

Weather events; traffic demand; and traffic incident occurrence, type, and location have both systematic and random elements. To the extent practical, the reliability methodology accounts for the systematic variation component in its predictive models. Specifically, it recognizes temporal changes in weather and traffic demand during the year, month, and day. It recognizes the influence of geographic location on weather and the influence of weather and traffic demand on incident occurrence.

A Monte Carlo approach uses essentially random inputs (within realistic limits) to model a system and produce probable outcomes.

Models of the systematic influences are included in the methodology. They are used to predict average weather, demand, and incident conditions during each analysis period. However, the use of averages to describe weather events and incident occurrence for such short time periods is counter to the objectives of reliability evaluation. The random element of weather events, demand variation, and traffic incident occurrence introduces a high degree of variability in the collective set of analysis periods that make up the reliability reporting period. Thus, replication of these random elements is important in any reliability evaluation. Monte Carlo methods are used for this purpose in the reliability methodology.

A random number seed is used with the Monte Carlo methods in the reliability methodology. A seed is used so that the sequence of random events can be reproduced. Unique seed numbers are separately established for weather events, demand variation, and incidents. For a given set of three seed numbers, a unique combination of weather events, demand levels, and incidents is estimated for each analysis period in the reliability reporting period.

One, two, or three of the seed numbers can be changed to generate a different set of conditions, if desired. For example, if the seed number for weather events is changed, a new series of weather events is created, and to the extent that weather influences incident occurrence, a new series of incidents is created. Similarly, the seed number for demand variation can be used to control whether a new series of demand levels is created. The seed number for incidents can be used to control whether a new series of incidents is created.

When alternatives are evaluated, the analyst will likely use one set of seed numbers as a variance reduction technique. In this application, the same seed numbers are used for all evaluations. With this approach, the results from an evaluation of one alternative can be compared with those from an evaluation of the baseline condition. Any observed difference in the results can be attributed to the changes associated with the alternative (i.e., they are not due to random changes in weather or incident events among the evaluations).

Confidence Intervals

A complete exploration of reliability would likely entail the use of multiple, separate evaluations of the same reliability reporting period, with each evaluation using a separate set of random number seeds. This approach may be particularly useful when the facility has infrequent weather events or incidents. With this approach, the evaluation is replicated multiple times, and the performance measures from each replication are averaged to produce a more reliable estimate of their long-run value. The confidence interval (expressed as a range) for the average produced in this manner can be computed with the following equation:

$$CI_{1-\alpha} = 2 \times t_{(1-\alpha/2),N-1} \times \frac{s}{\sqrt{N}}$$

Equation 17-3

where

$CI_{1-\alpha}$ = confidence interval for the true average value, with a level of confidence of $1 - \alpha$;

$t_{(1-\alpha),N-1}$ = Student's t-statistic for the probability of a two-sided error of α, with $N - 1$ degrees of freedom;

N = number of replications; and

s = standard deviation of the subject performance measure, computed by using results from the N replications.

The variable α equals the probability that the true average value lies outside the confidence interval. Values selected for α typically range from 0.05 (desirable) to 0.10. Selected values of Student's t-statistic are provided in Exhibit 17-17.

Exhibit 17-17
Student's t-Statistic

	Student's t-Statistic for Two Values of α	
Number of Replications	$\alpha = 0.05$	$\alpha = 0.10$
3	4.30	2.92
4	3.18	2.35
5	2.78	2.13
10	2.26	1.83
15	2.14	1.76
30	2.05	1.70

Scenario Sampling

Typical combinations of reliability reporting period, analysis period, weather event occurrence, and incident event occurrence could lead to a large number of unique scenarios. If the time required to evaluate all of these scenarios is considered too great for some reliability applications, a scenario sampling approach can be used.

The scenario sampling technique is used to minimize the total evaluation time while maintaining the underlying distribution of event occurrence. The analyst will need to input the scenario evaluation interval I. This interval has units of "days." The analyst can choose to evaluate every scenario for every day (i.e., input "1"). Alternatively, the analyst can choose to evaluate every scenario for every other day (i.e., input "2"). More generally, the analyst can input any integer number for the evaluation interval. Any number that is larger than "1" indicates that the evaluation will include only a sample of the total number of scenarios.

The evaluation interval is checked to ensure that all days in the reliability reporting period are equally sampled. In this manner, the subset of scenarios to be evaluated is not biased to include more of one weekday than another. The check examines the pattern produced by the input "days of week considered" D and evaluation interval I. An interval factor F is computed [$F = I - \text{int}(I/D) \times D$]. If five or seven days of the week are considered, values of I that yield $F > 0$ provide the desired representative sample. If two, three, four, or six days of the week are considered, values of I that yield $F = 1$ or $F = D - 1$ provide the desired sample. Any value of I that does not meet these conditions should be avoided because it will yield a scenario sample that is biased toward a specific weekday.

USE CASES

Travel time reliability measures can be used for a number of planning and roadway operating agency applications, including those given in Exhibit 17-18.

Each of the applications listed in Exhibit 17-18 has several potential uses for travel time reliability. For example, reliability may be assessed for existing or future facilities to identify current problem spots and future deficiencies in system operation. Reliability may provide additional performance measures that can be used in generating and evaluating alternatives. Reliability may supplement conventional measurements for prioritizing improvement projects.

Exhibit 17-18
Use Cases for Travel Time Reliability

Application	Use Cases for Travel Time Reliability
Long-range transportation plan Transportation improvement program Corridor or area plans Major investment studies Congestion management Operations planning	• Identifying *existing* facilities not meeting reliability standards • Identifying *future* facilities not meeting reliability standards • Generating alternatives to address reliability problems • Evaluating reliability benefits of improvement alternatives • Prioritizing operational improvements and traditional capacity improvements • Evaluating the probability of achieving acceptable reliability and LOS
Long-range planning: demand forecasting	• Improving modeling of destination, time of day, mode, and route choice

Planning has traditionally focused on capacity improvements and has been relatively insensitive to the reliability improvements that come with operations improvements. Thus, reliability can become an important new measure in identifying improvement alternatives, evaluating their benefits, and prioritizing them more accurately in relation to conventional capacity improvements.

Reliability adds another dimension of information on facility performance that can aid travel demand models in predicting the conditions under which people will choose to pay a toll for more reliable service. Reliability will enable better destination, time-of-day, mode, and route choice models.

Use Case No. 1: Detecting Existing Deficiencies

This use case for the reliability methods in the HCM involves monitoring conditions on a facility, identifying unacceptable performance, and detecting the primary causes of unreliable facility operation. It involves selecting the appropriate study period, performance measures, and thresholds of acceptance; calibrating the HCM operations models; and expanding limited data to a full reliability dataset.

Use Case No. 2: Forecasting Problems

This use case evaluates future reliability conditions on a facility, including the following:

- Expanding average annual (daily, peak period, or peak hour) volumes (forecast demand) to the full variety of study period demands,
- Estimating facility travel times by time slice within the full study period, and
- Comparing future with existing performance and identifying "significant" changes in performance.

The following are among the forecasting questions that Use Case No. 2 addresses:

1. How to forecast weather:
   a. Use of Monte Carlo or expected value techniques to forecast the frequency of future weather events.
   b. Number of years that the forecast must be carried into the future to obtain a reasonably likely set of scenarios.
2. How to forecast incident frequency:
   a. Use of Monte Carlo or expected value techniques.
   b. Number of future years that must be forecast to obtain a reasonably likely set of scenarios.
   c. Prediction of the effect of capacity improvements, demand changes, and ATDM improvements on crash frequencies.
3. Dealing with congestion overflows (e.g., over the entry link, over the last analysis period) when performance measures are computed and compared with existing conditions.
4. Calibrating this chapter's forecast reliability for future conditions to field-measured reliability under existing conditions (for data-rich agencies).

Use Case No. 3: Generating Alternatives

This use case identifies alternative operational and capacity improvements for addressing reliability problems. Selection of operational and capacity improvements that are likely to be best in addressing the primary causes of reliability problems on the facility is included.

This case requires that the analyst

1. Determine that a reliability problem exists (see Use Case No. 6),
2. Diagnose the causes of the reliability problem, and
3. Identify promising treatment options for addressing the problem.

As part of the diagnostic process, the analyst needs to be able to identify the facility's primary causes of unreliability and then identify two or three courses of action to address those causes. This approach requires guidance linking causes of unreliability to cost-effective solutions that can be considered.

Use Case No. 4: Reliability Benefits of Alternatives

This use case computes the reliability effects of alternative operational and capacity improvements for addressing reliability problems, including traditional capacity improvements as well as more innovative ATDM measures.

While Use Case No. 3 was primarily about diagnosis, Use Case No. 4 focuses on evaluating candidate treatment options. The analyst fleshes out possible treatments, estimates their effectiveness, and estimates their costs. This analysis requires procedures and parameters for computing the effects of capacity, operational, and ATDM improvements on existing or predicted reliability.

Once an agency has performed enough of these analyses, it can probably develop its own Case No. 3 diagnosis chart with locally specific treatment options.

Use Case No. 5: Prioritizing Improvements

This use case applies reliability performance measures in combination with other performance measures to prioritize investments in operational and capacity improvements. Estimation of the relative values of mean travel time improvements and travel time reliability improvements is included in this case.

While the reliability methodology provides results for only one facility at a time, agencies putting together a regional program will want to combine the results of individual facility analyses (freeways and urban streets) into a prioritized table. In essence, the issue is how to weight the relative benefits of reliability improvements versus more traditional capacity improvements. How much is average travel time worth to the agency and the public, compared with 95th percentile travel time or some other measure of reliability?

Use Case No. 6: Achieving Acceptable Performance

This use case estimates the probability of failure or the probability of achieving acceptable performance. Performance may be reported as achieving a minimum acceptable LOS.

This use defines and determines acceptable and unacceptable reliability performance. Thus, it is a critical input to the diagnostic process of Use Case No. 3. No diagnosis is needed when it is determined that no reliability problem exists. However, if Use Case No. 6 determines that a problem exists, Use Case No. 3 is used to diagnose the causes and identify promising treatment options.

Use Case No. 6 shares much with Case No. 5, but it introduces a new concept, acceptability or failure. The numerical results produced in Use Case No. 5 are compared with some standard—a national, state, or agency-specific standard of acceptable performance.

This use case introduces the concept of defining a standard both as a minimum acceptable performance level (such as LOS or PTI) and as the probability of failing to achieve that level (i.e., probability of failure). The standard is thus defined in two dimensions: a value and a probability of exceeding that value.

Use Case No. 5 deals with numerical outputs that are compared with each other (relativistic evaluation). In contrast, Use Case No. 6 compares the numerical outputs with an absolute standard (failure analysis).

Use Case No. 7: Improved Demand Modeling

This use case applies HCM methods to develop volume–reliability curves by facility type for use in a demand modeling environment to estimate reliability and to improve destination, time-of-day, mode choice, and route choice models.

USE OF ALTERNATIVE TOOLS

In some cases, a finer temporal sensitivity to dynamic changes in the system will be required for a reliability analysis than can be provided by the typical 15-min analysis period used by HCM methods. This situation may occur in evaluating traffic-responsive signal timing, traffic adaptive control, dynamic ramp metering, dynamic congestion pricing, or measures affecting the prevalence or duration of incidents with less than 10-min durations. There may also be scenarios and configurations that the HCM cannot address, such as cyclic spillback or adaptive signal control.

For such situations, this chapter's conceptual framework for evaluating travel time reliability can be applied to alternative analysis tools. The same conceptual approach—generating scenarios, assigning scenario probabilities, evaluating scenario performance, and summarizing the results—applies when alternative analysis tools, such as microsimulation, are used to estimate the reliability effects of operations improvements.

Before embarking on the use of alternative tools for reliability analysis, the analyst should consider the much greater analytical demands imposed by a reliability analysis following this chapter's conceptual analysis framework. Thousands of scenarios may need to be analyzed with the alternative tool in addition to the number of replications per scenario required by the tool itself to establish average conditions. Extracting and summarizing the results from numerous applications of the alternative tool may be a significant task.

If a microscopic simulation tool is used, some portions of this chapter's analysis framework that were fit to the HCM's 15-min analysis periods and tailored to the HCM's speed–flow curves will no longer be needed:

- Scenarios may be defined differently from and may be of longer or shorter duration than those used in HCM analysis.

- Incident start times and durations will no longer need to be rounded to the nearest 15-min analysis period.
- Weather start times and durations will no longer need to be rounded to the nearest 15-min analysis period.
- Demand will no longer need to be held constant for the duration of the 15-min analysis period.
- The peak hour factors used to identify the peak 15-min flow rate within the hour will no longer be applied. They will be replaced with the analysis tool's built-in randomization process.
- The reliability methodology's recommended saturation flow rate and free-flow speed adjustments for weather events and incidents will have to be converted by the analyst to the microsimulation model equivalents: desired speed distribution and desired headway distribution.
- Acceleration and deceleration rates will also be affected for some weather events.

If a less disaggregate tool is used (e.g., mesoscopic simulation analysis tool, dynamic traffic assignment tool, demand forecasting tool), many of this chapter's adaptations of the conceptual analysis framework to the HCM may still be appropriate or may need to be aggregated further. The analyst should consult the appropriate tool documentation and determine what further adaptations of the conceptual analysis framework might be required to apply the alternative tool to reliability analysis.

6. REFERENCES

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

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CHAPTER 18
URBAN STREET SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

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

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

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

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

A street segment's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road-user group. Performance measures cited in this chapter include motorized vehicle travel speed, motorized vehicle stop rate, automobile traveler perception score, pedestrian travel speed, pedestrian space, pedestrian level-of-service (LOS) score, bicycle travel speed, bicycle LOS score, transit vehicle travel speed, transit wait–ride score, and transit passenger LOS score.

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

CHAPTER ORGANIZATION

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

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

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

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

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

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

RELATED HCM CONTENT

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

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

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

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

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

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

2. CONCEPTS

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

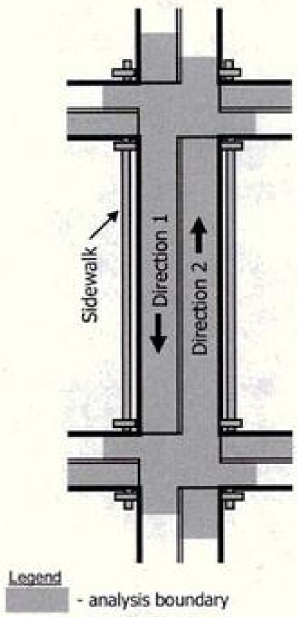
ANALYSIS TYPE

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

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

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

Spatial and Temporal Limits



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

Analysis Boundaries

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

Travel Directions to Be Evaluated

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

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

Study Period and Analysis Period

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

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

URBAN STREET SEGMENT DEFINED

Terminology

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

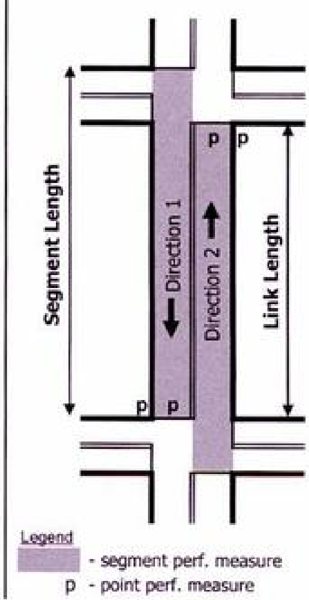
Points and Segments

A segment performance measure combines link performance and point performance.

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

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

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



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

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

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

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

LOS CRITERIA

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

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.

Motorized Vehicle Mode

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

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

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

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

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

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

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

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

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

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

Exhibit 18-1
LOS Criteria: Motorized Vehicle Mode

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

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

Pedestrian, Bicycle, and Transit Modes

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

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

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

Exhibit 18-2
LOS Criteria: Pedestrian Mode

Segment-Based Pedestrian LOS Score	Segment-Based LOS by Average Pedestrian Space (ft²/p) >60	>40–60	>24–40	>15–24	>8.0–15[a]	≤8.0[a]	Link-Based Pedestrian LOS: Link-Based LOS Score	LOS
≤2.00	A	B	C	D	E	F	≤1.50	A
>2.00–2.75	B	B	C	D	E	F	>1.50–2.50	B
>2.75–3.50	C	C	C	D	E	F	>2.50–3.50	C
>3.50–4.25	D	D	D	D	E	F	>3.50–4.50	D
>4.25–5.00	E	E	E	E	E	F	>4.50–5.50	E
>5.00	F	F	F	F	F	F	>5.50	F

Note: [a] In cross-flow situations, the LOS E/F threshold is 13 ft²/p. Chapter 4 describes the concept of "cross flow" and situations where it should be considered.

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

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

Exhibit 18-3
LOS Criteria: Bicycle and Transit Modes

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

The association between LOS score and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip along an urban street. The letter A was used to represent the best quality of service, and the letter F was used to represent the worst quality of service. "Best" and "worst" were left undefined, allowing the respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

SCOPE OF THE METHODOLOGIES

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

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

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

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

3. MOTORIZED VEHICLE METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

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

Spatial and Temporal Limits

Analysis Boundaries

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

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

Study Period and Analysis Period

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

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

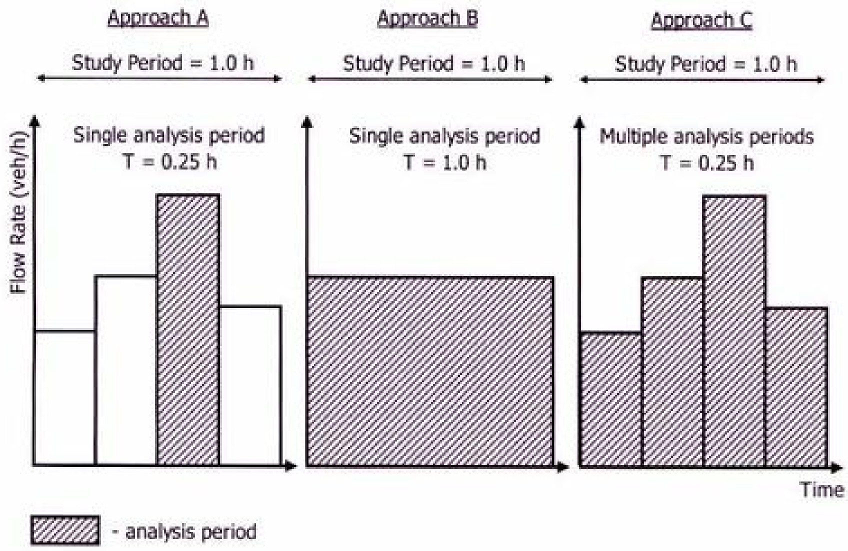


Exhibit 18-4
Three Alternative Study Approaches

Approach A is based on evaluation of the peak 15-min period during the study period. The analysis period *T* is 0.25 h. The equivalent hourly flow rate in vehicles per hour (veh/h) used for the analysis is based on either (*a*) a peak 15-min traffic count multiplied by four or (*b*) a 1-h demand volume divided by the peak hour factor. The former option is preferred for existing conditions when traffic counts are available; the latter option is preferred when hourly volumes are projected or when hourly projected volumes are added to existing volumes. Additional discussion on use of the peak hour factor is provided in the subsection titled Required Data and Sources.

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

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

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

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

Multiple Time Period Analysis

If the analysis period's demand volume exceeds capacity, a multiple time period analysis should be undertaken in which the study period includes an initial analysis period with no initial queue and a final analysis period with no residual queue. On a movement-by-movement and intersection-by-intersection basis, the initial queue for the second and subsequent periods is equal to the residual queue from the previous period. This approach provides a more accurate estimate of the delay associated with the congestion.

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

Segment Length Considerations

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

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

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

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

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

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

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

Performance Measures

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

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

Limitations of the Methodology

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

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

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

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

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

Lane Groups and Movement Groups

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

The motorized vehicle methodology in Chapter 19 is designed to evaluate the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group*. In general, a separate lane group is established for (*a*) each lane (or combination of adjacent lanes) that exclusively serves one movement and (*b*) each lane shared by two or more movements.

The concept of *movement groups* is established to facilitate data entry to the methodology. In this regard, input data describing intersection traffic are traditionally specific to the movement (e.g., left-turn movement volume) and not specific to the lane (e.g., analysts rarely have the volume for a lane shared by left-turning and through vehicles). A separate movement group is established for (*a*) each turn movement with one or more exclusive turn lanes and (*b*) the through movement (inclusive of any turn movements that share a lane).

REQUIRED DATA AND SOURCES

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

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

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

The data elements listed in Exhibit 18-5 do not include variables that are considered to represent calibration factors (e.g., acceleration rate). A calibration factor typically has a relatively narrow range of reasonable values or has a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Traffic Characteristics Data

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

Demand Flow Rate

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

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

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

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

Access Point Flow Rate

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

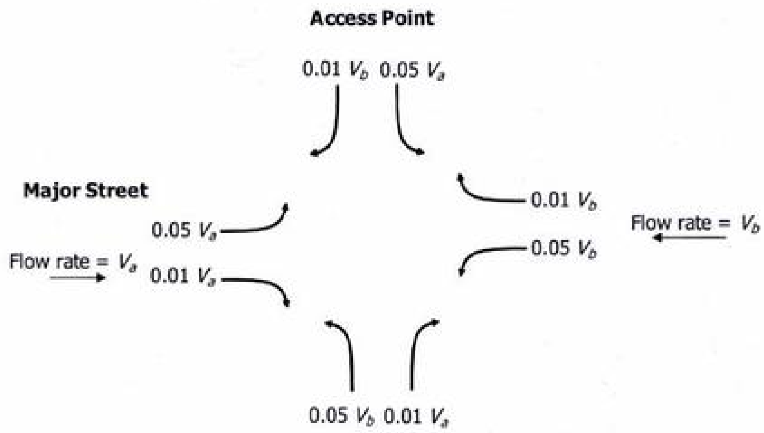
This flow rate is needed for all movements on each "active" access point approach and for all major-street movements at the intersection with one or more "active" access point approaches. An access point approach is considered to be *active* if its volume is sufficient to have some impact on segment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

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

If a planning analysis is being conducted in which (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, each movement's hourly demand should be divided by the intersection peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor should be based on local traffic peaking trends. If a local factor is not available, the default value in Exhibit 19-11 of Chapter 19 can be used.

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

Exhibit 18-6
Default Turn Proportions for Access Point Intersections



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

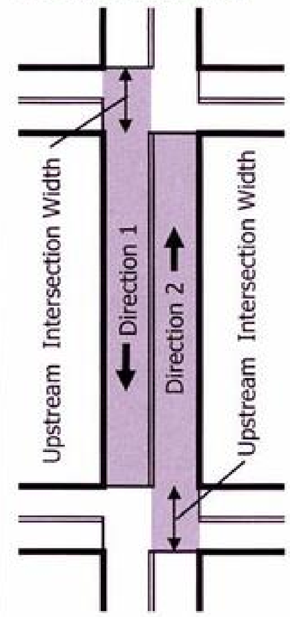
Midsegment Flow Rate

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

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

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

Two-Way Vehicular Travel



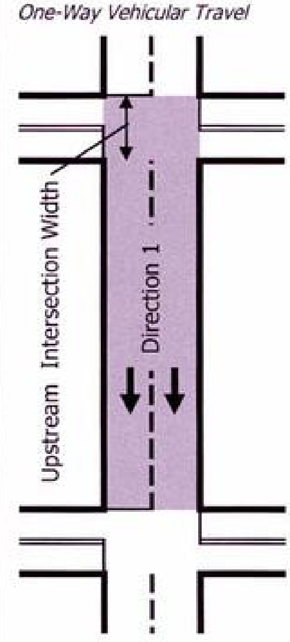
Geometric Design Data

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

Number of Lanes at Boundary Intersection

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

One-Way Vehicular Travel



Upstream Intersection Width

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

Turn Bay Length at Boundary Intersection

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

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

Number of Through Lanes on the Segment

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

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

Number of Lanes at Access Points

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

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

Turn Bay Length at Access Points

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

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

Segment Length

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

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

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

Restrictive Median Length

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

Proportion of Segment with Curb

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

Number of Access Point Approaches

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

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

Equation 18-1

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

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

Exhibit 18-7
Default Access Point Density Values

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

Proportion of Segment with On-Street Parking

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

Other Data and Performance Measures

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

Analysis Period Duration

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

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

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

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

Speed Limit

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

Through Control Delay

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

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

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

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

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

Capacity

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

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

Equation 18-2

where

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

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

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

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

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

Midsegment Delay and Stops

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

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

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

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

OVERVIEW OF THE METHODOLOGY

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

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

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

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

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

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

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

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

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

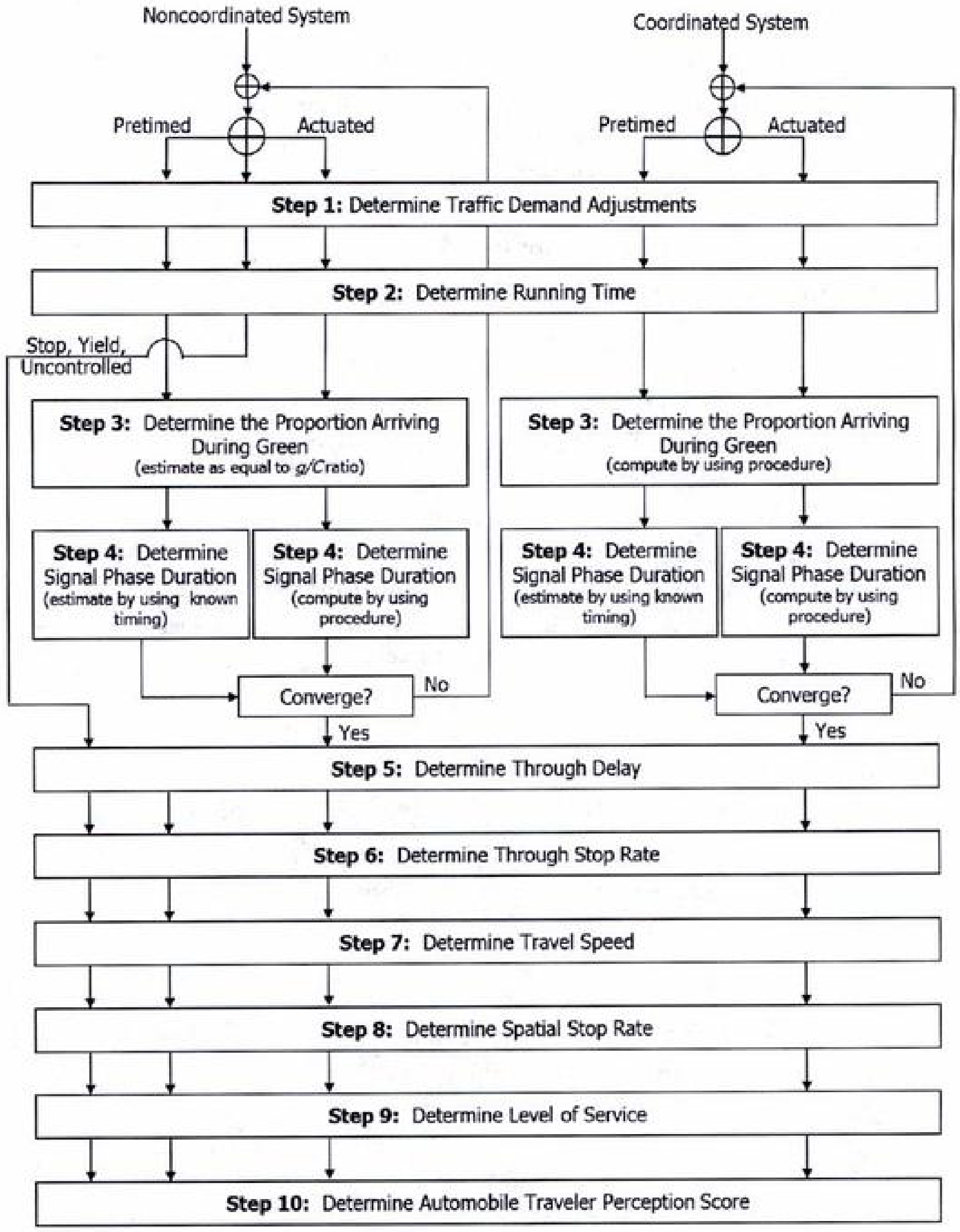
Details on the methodology for segments with roundabouts as boundary intersections can be found in Section 9 of Chapter 30.

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

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

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

Exhibit 18-8
Motorized Vehicle Methodology for Urban Street Segments



COMPUTATIONAL STEPS

Step 1: Determine Traffic Demand Adjustments

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

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

Capacity Constraint

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

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

Volume Balance

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

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

Origin–Destination Distribution

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

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

Exhibit 18-9
Entry and Exit Volume on Example Segment

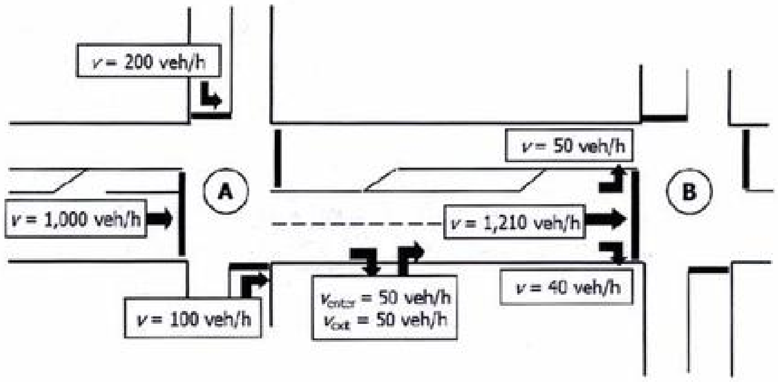


Exhibit 18-10
Example Origin–Destination Distribution Matrix

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

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

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

Spillback Occurrence

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

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

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

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

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

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

Step 2: Determine Running Time

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

A. Determine Free-Flow Speed

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

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

Base Free-Flow Speed

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

Equation 18-3

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

where

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

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

S_0 = speed constant (mi/h),

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

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

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

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

Exhibit 18-11
Base Free-Flow Speed Adjustment Factors

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

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

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

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

Adjustment for Signal Spacing

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

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

Equation 18-4

where

f_L = signal spacing adjustment factor,

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

L_s = distance between adjacent signalized intersections (ft).

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

Free-Flow Speed

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

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

Equation 18-5

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

B. Compute Adjustment for Vehicle Proximity

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

Equation 18-6

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

where

f_v = proximity adjustment factor,

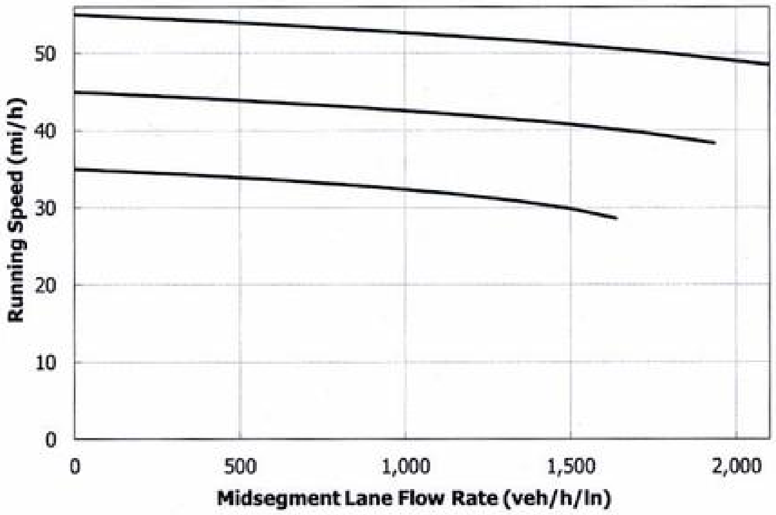
v_m = midsegment demand flow rate (veh/h),

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

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

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

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



C. Compute Delay due to Turning Vehicles

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

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

Exhibit 18-13
Delay due to Turning Vehicles

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

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

D. Estimate Delay due to Other Sources

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

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

E. Compute Segment Running Time

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

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

Equation 18-7

with

Equation 18-8

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

where

t_R = segment running time (s);

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

L = segment length (ft);

f_x = control-type adjustment factor;

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

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

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

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

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

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

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

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

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

Step 3: Determine the Proportion Arriving During Green

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

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

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

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

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

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

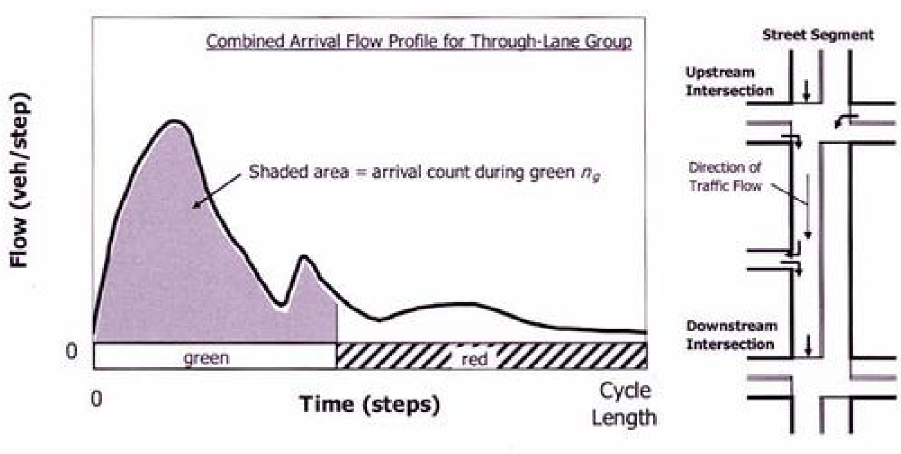


Exhibit 18-14
Use of an Arrival Flow Profile to Estimate the Volume Arriving During Green

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

Equation 18-9

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

where

P = proportion of vehicles arriving during the green indication,

n_g = arrival count during green (veh),

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

C = cycle length (s).

Step 4: Determine Signal Phase Duration

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

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

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

Step 5: Determine Through Delay

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

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

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

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

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

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

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

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

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

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

Equation 18-10

where

d_t = through delay (s/veh),

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

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

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

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

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

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

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

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

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

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

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

Step 6: Determine Through Stop Rate

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

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

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

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

Equation 18-11

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

with

Equation 18-12

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

Equation 18-13

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

Equation 18-14

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

where

- h = full stop rate (stops/veh),
- N_f = number of fully stopped vehicles (veh/ln),
- g = effective green time (s),
- s = adjusted saturation flow rate (veh/h/ln),
- Q_{2+3} = back-of-queue size (veh/ln),

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Step 7: Determine Travel Speed

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

Equation 18-15

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

where

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

L = segment length (ft),

t_R = segment running time (s), and

d_t = through delay (s/veh).

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

Step 8: Determine Spatial Stop Rate

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

Equation 18-16

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

where

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

h = full stop rate (stops/veh),

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

L = segment length (ft).

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

Step 9: Determine LOS

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

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

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

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

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

Step 10: Determine Automobile Traveler Perception Score

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

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

Equation 18-17

with

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

Equation 18-18

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

Equation 18-19

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

Equation 18-20

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

Equation 18-21

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

Equation 18-22

where

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

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

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

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

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

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

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

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

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

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

4. PEDESTRIAN METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

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

Spatial Limits

Travel Directions to Be Evaluated

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

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

Segment-Based Evaluation

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

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

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

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

Link-Based Evaluation

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

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

Performance Measures

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

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

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

Exhibit 18-15
Qualitative Description of Pedestrian Space

Pedestrian Space (ft^2/p) Random Flow	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 18-15 indicate a sensitivity to flow condition. Random pedestrian flow is typical of most segments. Platoon flow is appropriate for shorter segments (e.g., in downtown areas) with signalized boundary intersections.

Limitations of the Methodology

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

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

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

REQUIRED DATA AND SOURCES

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

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

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

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

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

Traffic Characteristics Data

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

Pedestrian Flow Rate

The pedestrian flow rate is based on the count of pedestrians traveling along the outside of the subject segment during the analysis period. A separate count is taken for each direction of travel along the side of the segment. Each count is divided by the analysis period duration to yield a directional hourly flow rate. These rates are then added to obtain the pedestrian flow rate for that side.

Proportion of On-Street Parking Occupied

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

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

Geometric Design Data

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

Downstream Intersection Width

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

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

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

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

Curb Presence

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

Presence of a Sidewalk

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

Total Walkway Width

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

Effective Width of Fixed Objects

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

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

Buffer Width and Spacing of Objects in Buffer

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

Other Data

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

Distance to Nearest Signal-Controlled Crossing

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

Legality of Midsegment Pedestrian Crossing

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

Proportion of Sidewalk Adjacent to Window, Building, or Fence

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

Performance Measures

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

Motorized Vehicle Running Speed

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

Pedestrian Delay

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

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

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

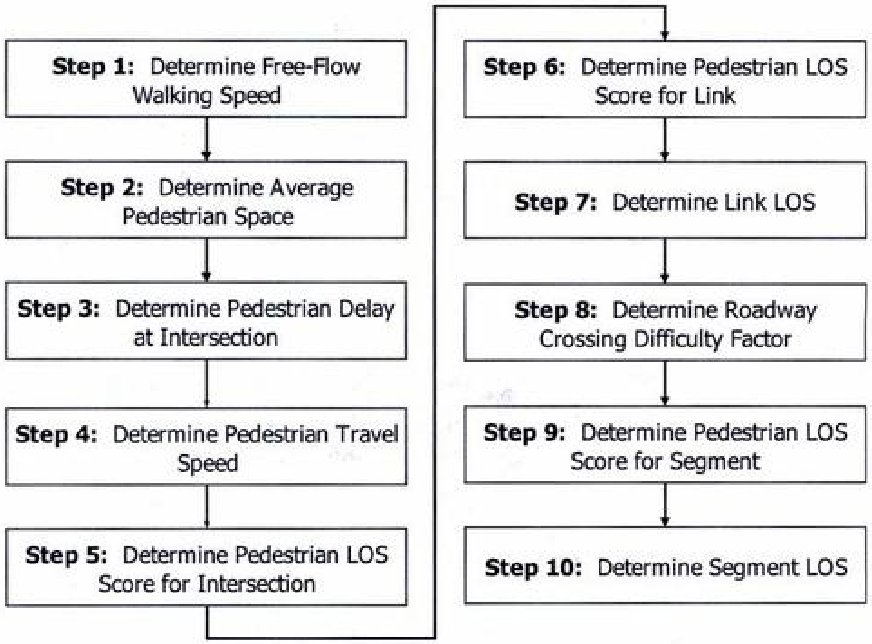
Pedestrian LOS Score for Intersection

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

OVERVIEW OF THE METHODOLOGY

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

Exhibit 18-17
Pedestrian Methodology for Urban Street Segments



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

COMPUTATIONAL STEPS

Step 1: Determine Free-Flow Walking Speed

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

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

Step 2: Determine Average Pedestrian Space

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

A. Compute Effective Sidewalk Width

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

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

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

Equation 18-23

with

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

Equation 18-24

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

Equation 18-25

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

Equation 18-26

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

Equation 18-27

where

W_E = effective sidewalk width (ft),

W_T = total walkway width (ft),

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

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

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

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

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

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

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

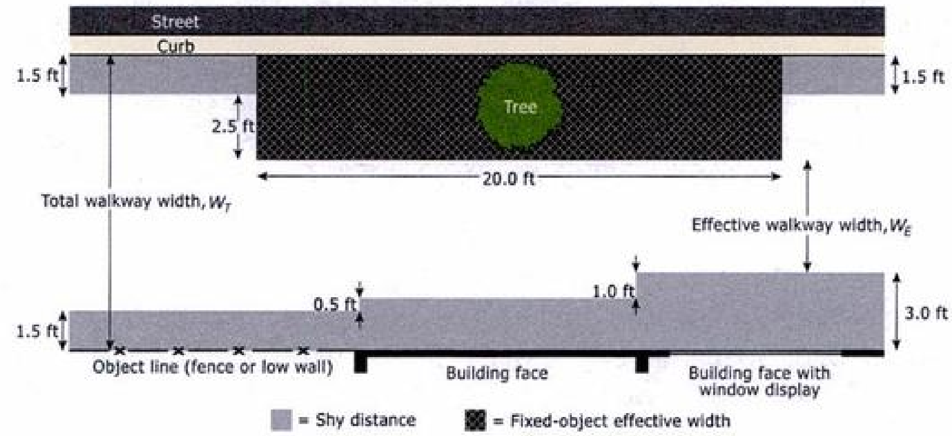
p_{fence} = proportion of sidewalk length adjacent to a fence or low wall (decimal),

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

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

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

Exhibit 18-18
Width Adjustments for Fixed Objects



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

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

B. Compute Pedestrian Flow Rate per Unit Width

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

Equation 18-28

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

where

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

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

W_E = effective sidewalk width (ft).

C. Compute Average Walking Speed

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

Equation 18-29

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

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

D. Compute Pedestrian Space

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

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

Equation 18-30

where A_p is the pedestrian space (ft^2/p) and all other variables are as previously defined.

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

Step 3: Determine Pedestrian Delay at Intersection

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

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

Step 4: Determine Pedestrian Travel Speed

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

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

Equation 18-31

where

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

L = segment length (ft),

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

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

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

Step 5: Determine Pedestrian LOS Score for Intersection

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

Step 6: Determine Pedestrian LOS Score for Link

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

Equation 18-32
$$I_{p,\text{link}} = 6.0468 + F_w + F_v + F_S$$

with

Equation 18-33
$$F_w = -1.2276 \ln\left(W_v + 0.5\, W_l + 50\, p_{pk} + W_{buf} f_b + W_{aA} f_{sw}\right)$$

Equation 18-34
$$F_v = 0.0091 \frac{v_m}{4\, N_{th}}$$

Equation 18-35
$$F_S = 4\left(\frac{S_R}{100}\right)^2$$

where

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

F_w = cross-section adjustment factor;

F_v = motorized vehicle volume adjustment factor;

F_S = motorized vehicle speed adjustment factor;

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

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

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

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

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

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

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

W_T = total walkway width (ft);

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

f_{sw} = sidewalk width coefficient = $6.0 - 0.3\, W_{aA}$;

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

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

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

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

Exhibit 18-19
Variables for Pedestrian LOS Score for Link

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

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

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

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

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

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

Step 7: Determine Link LOS

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

Step 8: Determine Roadway Crossing Difficulty Factor

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

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

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

Each of these two crossing options is considered in this step, with the option requiring the least 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

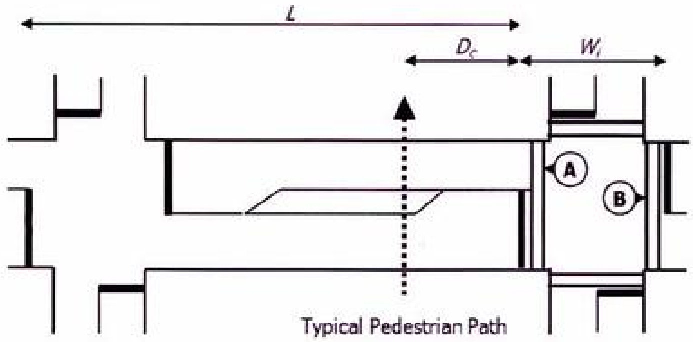
The delay incurred as a consequence of diverting to the nearest signal-controlled crossing is computed first. It includes the delay involved in walking to and from the midsegment crossing point to the nearest signal-controlled crossing and the delay waiting to cross at the signal. Hence, calculation 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 18-20. Exhibit 18-20(a) illustrates the distance D_c when the pedestrian diverts to the nearest signalized intersection. This distance is measured from the crossing location to the signalized intersection.

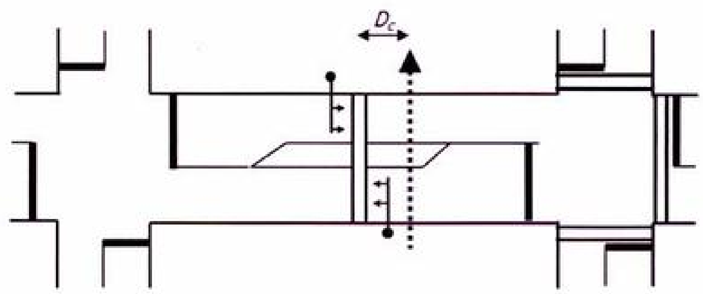
Exhibit 18-20(b) illustrates the distance D_c when a signalized crosswalk is provided at a midsegment location. In this situation, the distance is measured from the pedestrian crossing location to the location of the signalized crosswalk. In either case, the distance D_c is an input value provided by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

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

Exhibit 18-20
Diversion Distance Components



(a) Divert to Nearest Boundary Intersection



(b) Divert to Midsegment Signalized Crosswalk

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

Equation 18-36

$$D_d = 2\, D_c$$

where

D_d = diversion distance (ft), and

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

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

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

Equation 18-37

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

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 incurred in crossing the segment at the nearest signal-controlled crossing (s/p).

The pedestrian delay incurred in 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 18-38.

Equation 18-38

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

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),

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

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

If the factor obtained from Equation 18-38 is less than 0.80, the factor is set equal to 0.80. If the factor is greater than 1.20, it is set equal to 1.20.

The pedestrian waiting delay was determined in Step 3. If a midsegment crossing is illegal, 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 with Equation 18-39:

Equation 18-39

$$I_{p,seg} = 0.75\left[\frac{\left(F_{cd}\,I_{p,link} + 1\right)^3 \frac{L}{S_p} + \left(I_{p,int} + 1\right)^3 d_{pp}}{\frac{L}{S_p} + d_{pp}}\right]^{\frac{1}{3}} + 0.125$$

where $I_{p,seg}$ is the pedestrian LOS score for the segment and all 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 18-2 to determine the LOS for the specified direction of travel along the subject segment. If a sidewalk does not exist and pedestrians are relegated to walking in the street, LOS is determined by using Exhibit 18-3 because the pedestrian space concept does not apply.

5. BICYCLE METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

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

Spatial Limits

Travel Directions to Be Evaluated

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

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

Segment-Based Evaluation

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

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

Link-Based Evaluation

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

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

Performance Measures

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

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

Limitations of the Methodology

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

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

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

REQUIRED DATA AND SOURCES

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

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

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

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

The data elements listed in Exhibit 18-21 do not include variables that are considered to represent calibration factors. A calibration factor typically has a relatively narrow range of reasonable values or has a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Traffic Characteristics Data

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

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

Geometric Design Data

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

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

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

Other Data

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

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

Exhibit 18-22
Pavement Condition Rating

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

Performance Measures

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

Motorized Vehicle Running Speed

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

Bicycle Delay

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

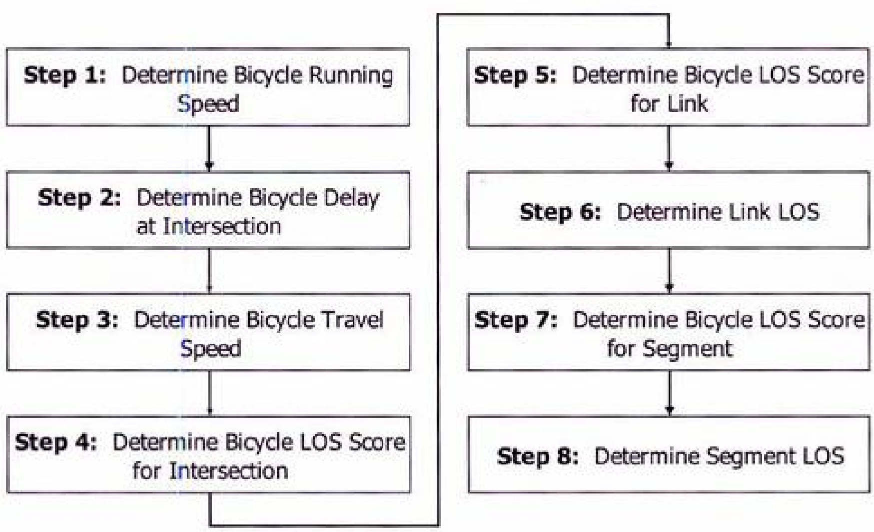
Bicycle LOS Score for Intersection

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

OVERVIEW OF THE METHODOLOGY

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

Exhibit 18-23
Bicycle Methodology for Urban Street Segments



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

COMPUTATIONAL STEPS

Step 1: Determine Bicycle Running Speed

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

Step 2: Determine Bicycle Delay at Intersection

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

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

Step 3: Determine Bicycle Travel Speed

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

Equation 18-40

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

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, the bicycle methodology described in Chapter 19 is used for this determination. If the boundary intersection is two-way STOP controlled, the score is equal to 0.0.

Step 5: Determine Bicycle LOS Score for Link

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

$$I_{b,\text{link}} = 0.760 + F_w + F_v + F_s + F_p$$ **Equation 18-41**

with

$$F_w = -0.005\, W_e^2$$ **Equation 18-42**

$$F_v = 0.507 \ln\left(\frac{v_{ma}}{4\, N_{th}}\right)$$ **Equation 18-43**

$$F_S = 0.199\,[1.1199 \ln(S_{Ra} - 20) + 0.8103](1 + 0.1038\, P_{HVa})^2$$ **Equation 18-44**

$$F_p = \frac{7.066}{P_c^2}$$ **Equation 18-45**

where

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

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_p = pavement condition adjustment factor,

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

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

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

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

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

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

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

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

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

Exhibit 18-24
Variables for Bicycle LOS Score for Link

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

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

Step 6: Determine Link LOS

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

Step 7: Determine Bicycle LOS Score for Segment

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

Equation 18-46

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

with

Equation 18-47

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

where

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

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

F_c = unsignalized conflicts factor;

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

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

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

Step 8: Determine Segment LOS

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

6. TRANSIT METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

Spatial Limits

Travel Directions to Be Evaluated

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

Route-Based Evaluation

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

Performance Measures

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

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

Limitations of the Methodology

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

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

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

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

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

REQUIRED DATA AND SOURCES

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

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

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

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

The data elements listed in Exhibit 18-25 do not include variables that are considered to represent calibration factors. A calibration factor typically has a relatively narrow range of reasonable values or has a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Traffic Characteristics Data

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

Dwell Time

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

Excess Wait Time

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

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

Passenger Trip Length

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

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

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

Transit Frequency

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

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

Passenger Load Factor

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

Geometric Design Data

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

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

Other Data

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

Area Type

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

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

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

Transit Stop Location

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

Transit Stop Position

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

Proportion of Stops with Shelters and with Benches

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

Performance Measures

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

Motorized Vehicle Running Time

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

Pedestrian LOS Score for Link

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

Reentry Delay

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

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

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

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

Volume-to-Capacity Ratio (If Roundabout)

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

OVERVIEW OF THE METHODOLOGY

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

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

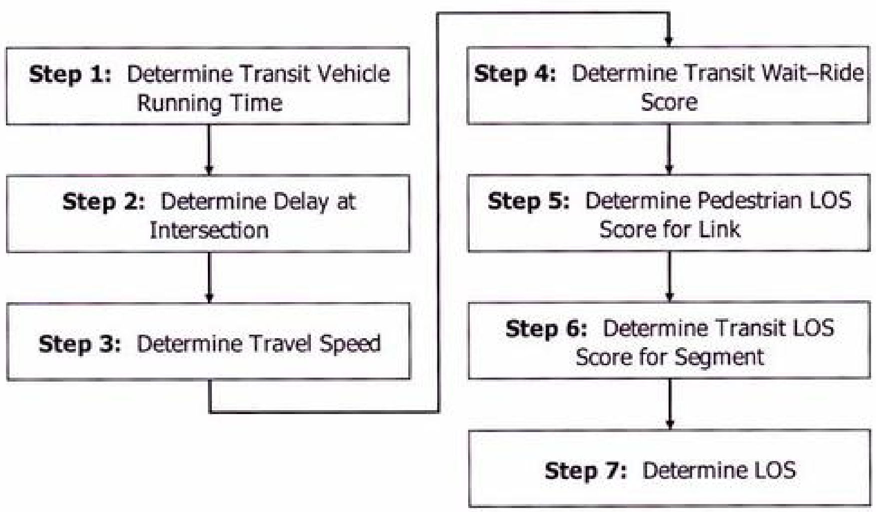


Exhibit 18-26
Transit Methodology for Urban Street Segments

COMPUTATIONAL STEPS

Step 1: Determine Transit Vehicle Running Time

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

A. Compute Segment Running Speed

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

Equation 18-48

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

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 18-7 in Step 2 of the motorized vehicle methodology.

B. Compute Delay due to a Stop

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

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

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

Acceleration–Deceleration Delay

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

Equation 18-49

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

with

Equation 18-50

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

where

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

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

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

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

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

g = effective green time (s), and

C = cycle length (s).

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

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

Delay due to Serving Passengers

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

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

Equation 18-51

where

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

t_d = average dwell time (s), and

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

Reentry Delay

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

Delay due to a Stop

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

Equation 18-52

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

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

C. Compute Segment Running Time

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

Equation 18-53

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

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

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

Step 2: Determine Delay at Intersection

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

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

Equation 18-54

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

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 18-54 is obtained from Exhibit 18-27.

Exhibit 18-27
Transit Vehicle Running Time Loss

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

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

Step 3: Determine Travel Speed

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

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

Equation 18-55

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

Step 4: Determine Transit Wait–Ride Score

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

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

A. Compute Headway Factor

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

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

Equation 18-56

where

F_h = headway factor, and

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

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

B. Compute Perceived Travel Time Factor

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

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

Equation 18-57

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

with

Equation 18-58

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

Equation 18-59

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

Equation 18-60

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

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 typically (mi);

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

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

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

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

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

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

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

Equation 18-61

where

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

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

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

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

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

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

C. Compute Wait–Ride Score

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

Equation 18-62

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

where

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

F_h = headway factor, and

F_{tt} = perceived travel time factor.

Step 5: Determine Pedestrian LOS Score for Link

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

Step 6: Determine Transit LOS Score for Segment

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

Equation 18-63

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

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

Step 7: Determine LOS

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

7. APPLICATIONS

EXAMPLE PROBLEMS

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

GENERALIZED DAILY SERVICE VOLUMES

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

ANALYSIS TYPE

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

Operational Analysis

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

Design Analysis

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

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

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

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

Planning and Preliminary Engineering Analysis

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

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

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

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

USE OF ALTERNATIVE TOOLS

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

Comparison of Motorized Vehicle Methodology and Alternative Tools

Motorized Vehicle Methodology

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

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

Alternative Tools

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

Conceptual Differences

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

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

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

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

Alternative Tool Application Guidance

Development of HCM-Compatible Performance Measures

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

Adjustment of Parameters

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

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

Sample Calculations Illustrating Alternative Tool Applications

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

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

8. REFERENCES

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

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5. U.S. Department of Transportation. *2004 Status of the Nation's Highways, Bridges, and Transit: Conditions and Performance*. Washington, D.C., 2004. http://www.fhwa.dot.gov/policy/olsp/reportspubs.htm. Accessed Sept. 4, 2008.
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CHAPTER 19
SIGNALIZED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

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

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

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

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

An intersection's performance is described by the use of one or more quantitative measures that characterize some aspect of the service provided to a specific road-user group. Performance measures cited in this chapter include volume-to-capacity ratio, motorized vehicle control delay, pedestrian corner circulation area, pedestrian delay, pedestrian level-of-service (LOS) score, bicycle delay, and bicycle LOS score.

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

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

CHAPTER ORGANIZATION

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

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

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

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

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

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

RELATED HCM CONTENT

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

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

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

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

2. CONCEPTS

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

TRAFFIC SIGNAL CONCEPTS

Types of Traffic Signal Control

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

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

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

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

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

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

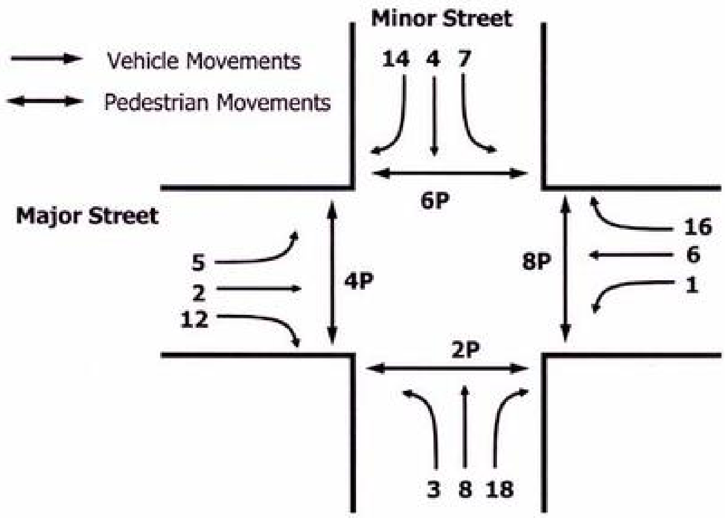
- *Coordinated-actuated control* is a variation of semiactuated operation. It uses the controller's force-off settings to constrain the noncoordinated phases associated with the minor movements so that the coordinated phases are served at the appropriate time during the signal cycle, and progression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

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

Intersection Traffic Movements

Exhibit 19-1 illustrates typical vehicle and pedestrian traffic movements at a four-leg intersection. Three vehicular traffic movements and one pedestrian traffic movement are shown for each intersection approach. Each movement is assigned a unique number or a number and letter combination. The letter "P" denotes a pedestrian movement. The number assigned to each left-turn and through movement is the same as the number assigned to each phase by National Electrical Manufacturers Association (NEMA) specifications.

Exhibit 19-1
Intersection Traffic Movements and Numbering Scheme



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

Signal Phase Sequence

Modern actuated controllers implement signal phasing by using a dual-ring structure that allows for the concurrent presentation of a green indication to two phases. Each phase serves one or more movements that do not conflict with each other. Early controllers used a single-ring structure in which all nonconflicting movements were assigned to a common phase, and its duration was dictated by the movement needing the most time. Of the two structures, the dual-ring structure is more efficient because it allows the controller to adapt phase duration and sequence to the needs of the individual movements. The dual-ring structure is typically used with eight phases; however, more phases are available for complex signal phasing. The eight-phase dual-ring structure is shown in Exhibit 19-2. The symbol Φ represents the word *phase*, and the number following the symbol represents the phase number.

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

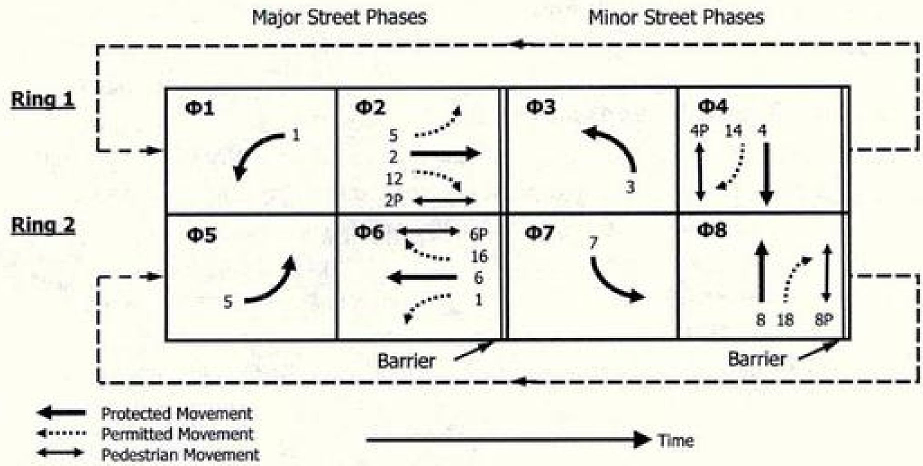


Exhibit 19-2 shows one way traffic movements can be assigned to each of the eight phases. These assignments are illustrative, but they are not uncommon. Each left-turn movement is assigned to an exclusive phase. During this phase, the left-turn movement is "protected," so it receives a green arrow indication. Each through, right-turn, and pedestrian movement combination is also assigned to an exclusive phase. The dashed arrows indicate turn movements that are served in a "permitted" manner so that the turn can be completed only after yielding the right-of-way to conflicting movements.

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

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

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

Operational Modes

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

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

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

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

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

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

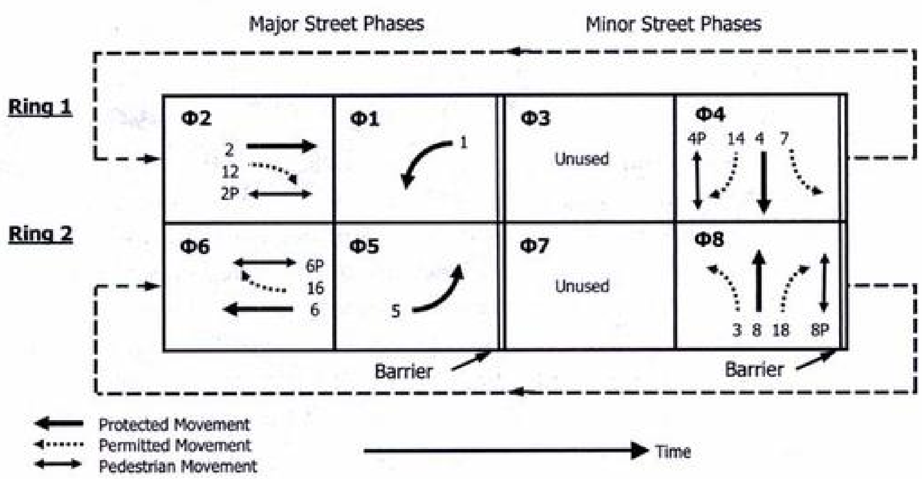
Left-Turn Phase Sequence

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

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

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

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



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

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

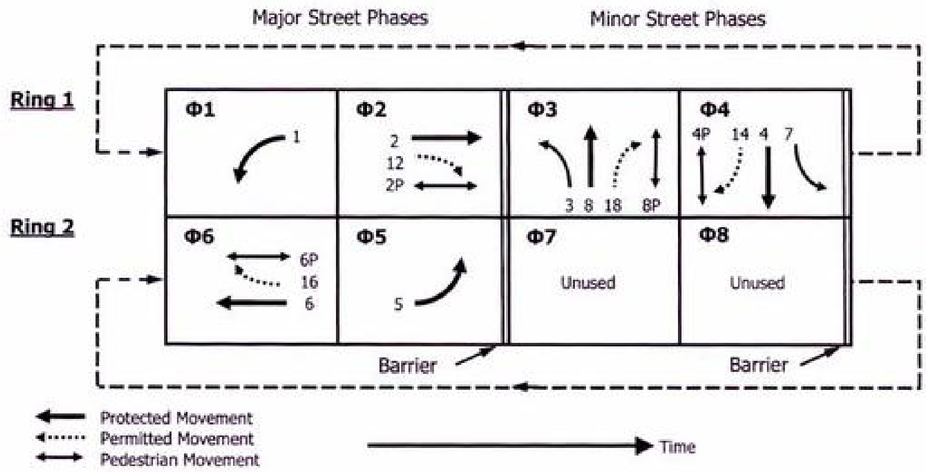


Exhibit 19-4
Illustrative Protected Lead–Lag and Split Phasing

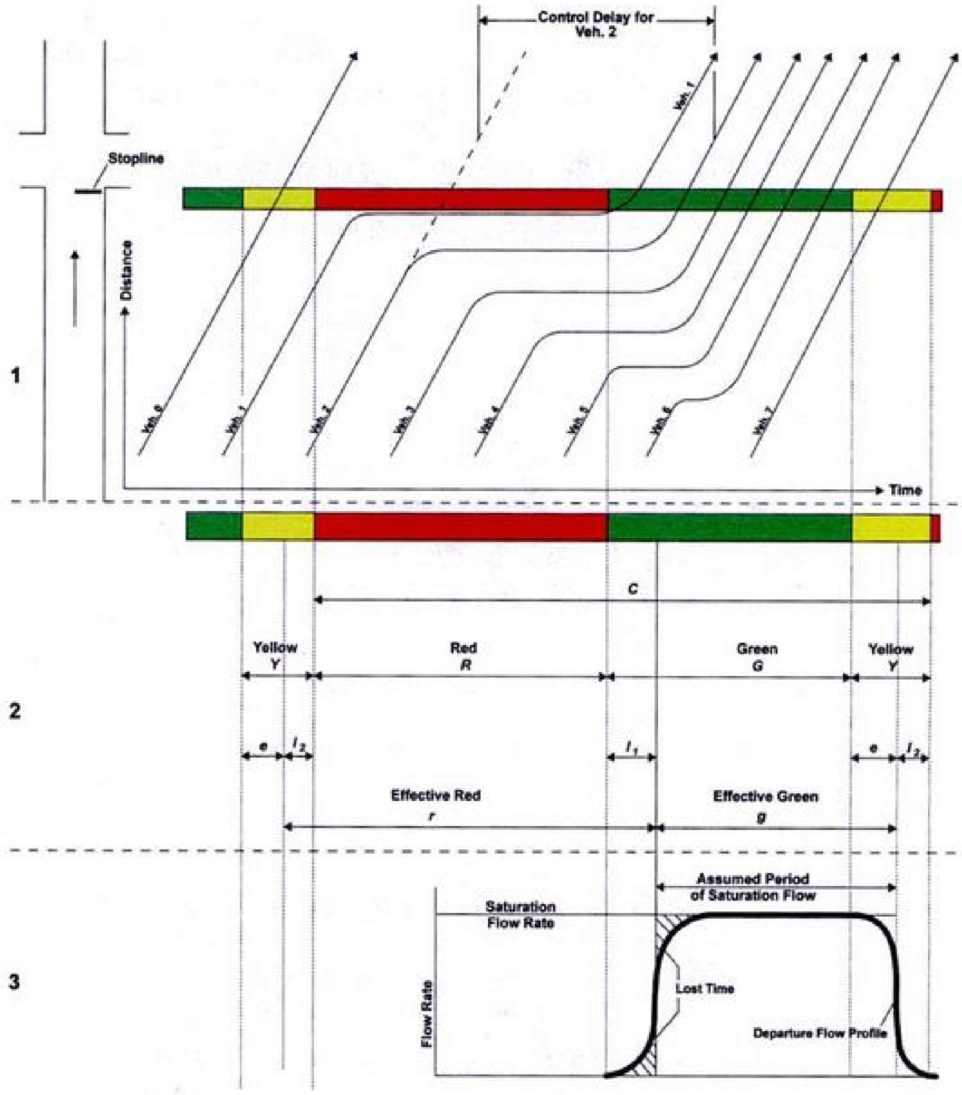
Traffic Flow Characteristics

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

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

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

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



Lost Time

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

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

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

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

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

Equation 19-1

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

where

l_t = phase lost time (s),

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

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

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

Y = yellow change interval (s), and

R_c = red clearance interval (s).

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

Saturation Flow Rate

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

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

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

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

Capacity

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

Phase Duration

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

The duration of an actuated phase is composed of five time periods. The first period represents the time lost while the queue reacts to the signal indication changing to green. The second interval represents the time required to clear the queue of vehicles. The third period represents the time the green indication is extended by randomly arriving vehicles. It ends when there is a gap in traffic (i.e., gap-out) or the green extends to the maximum limit (i.e., max-out). The fourth period represents the yellow change interval, and the fifth period represents the red clearance interval. The duration of an actuated phase is defined by Equation 19-2.

Equation 19-2

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

where

D_p = phase duration (s),

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

g_s = queue service time (s),

g_e = green extension time (s),

Y = yellow change interval (s), and

R_c = red clearance interval (s).

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

Exhibit 19-7
Time Elements Influencing Actuated Phase Duration

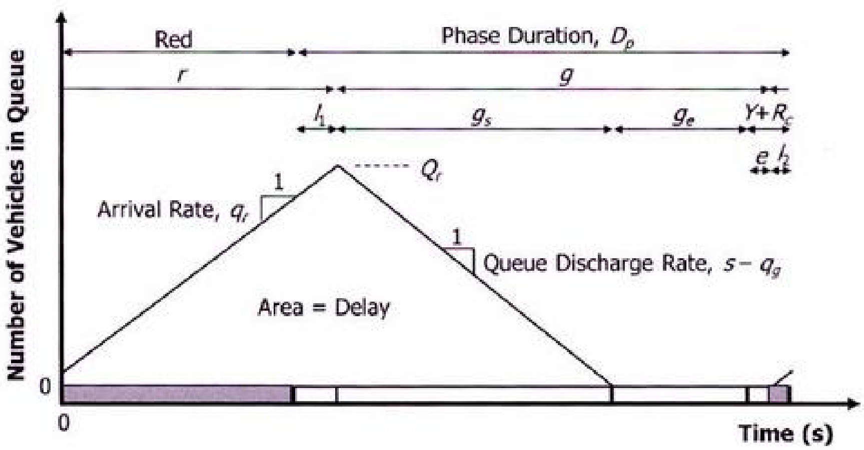


Exhibit 19-7 shows the relationship between phase duration and queue size for the average signal cycle. During the red interval, vehicles arrive at a rate of q_r and form a queue. The queue reaches its maximum size l_1 seconds after the red interval ends. At this time, the queue begins to discharge at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

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

Equation 19-3

$$g = D_p - l_1 - l_2$$

$$g = g_s + g_e + e$$

where

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

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

ANALYSIS TYPE

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

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

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

SPATIAL AND TEMPORAL LIMITS

Analysis Boundaries

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

Study Period and Analysis Period

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

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

LOS CRITERIA

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

Motorized Vehicle Mode

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

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

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

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

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

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

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

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

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

Exhibit 19-8 lists the LOS thresholds established for the motorized vehicle mode at a signalized intersection.

Exhibit 19-8
LOS Criteria: Motorized Vehicle Mode

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

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

Pedestrian and Bicycle Modes

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

Exhibit 19-9 lists the range of scores associated with each LOS for the pedestrian and bicycle travel modes. The association between score value and LOS is based on traveler perception research. Travelers were asked to rate the quality of service associated with a specific trip through a signalized intersection. The letter "A" was used to represent the best quality of service, and the letter "F" was used to represent the worst quality of service. *Best* and *worst* were left undefined, allowing respondents to identify the best and worst conditions on the basis of their traveling experience and perception of service quality.

Exhibit 19-9
LOS Criteria: Pedestrian and Bicycle Modes

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

SCOPE OF THE METHODOLOGIES

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

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

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

3. CORE MOTORIZED VEHICLE METHODOLOGY

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

SCOPE OF THE METHODOLOGY

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

- *Upstream intersections*. The influence of an upstream signalized intersection on the subject intersection's operation is addressed by input variables that simply, and subjectively, describe platoon structure and the uniformity of arrivals on a cyclic basis. Chapter 18, Urban Street Segments, extends the methodology described in this chapter to the evaluation of an intersection that is part of a coordinated signal system.
- *Target travel modes*. The motorized vehicle methodology addresses mixed automobile, motorcycle, truck, and transit traffic streams in which the automobile represents the largest percentage of all vehicles. The methodology is not designed to evaluate the performance of other types of vehicles (e.g., golf carts, motorized bicycles).

Spatial and Temporal Limits

Analysis Boundaries

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

Study Period and Analysis Period

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

Exhibit 19-10 demonstrates three alternative approaches an analyst might use for a given evaluation. Other alternatives exist, and the study period can exceed 1 h. Approach A is the approach that has traditionally been used and, unless otherwise justified, is the approach that is recommended for use.

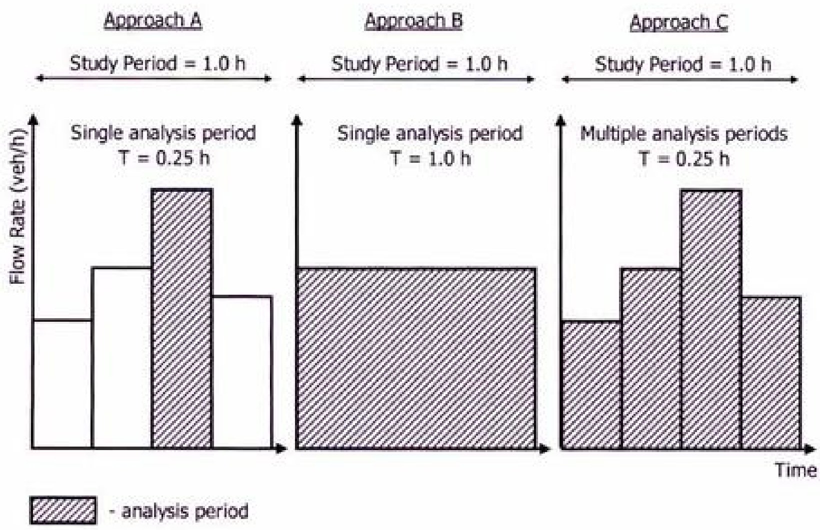


Exhibit 19-10
Three Alternative Study Approaches

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

Approach A is based on evaluation of the peak 15-min period during the study period. The analysis period T is 0.25 h. The equivalent hourly flow rate in vehicles per hour used for the analysis is based on either (*a*) a peak 15-min traffic count multiplied by four or (*b*) a 1-h demand volume divided by the peak hour factor. The former option is preferred for existing conditions when traffic counts are available; the latter option is preferred when hourly projected volumes are used or when hourly projected volumes are added to existing volumes. Additional discussion on use of the peak hour factor is provided in the subsection titled Required Data and Sources.

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

Approach C uses a 1-h study period and divides it into four 0.25-h analysis periods. This approach accounts for systematic flow rate variation among analysis periods. It also accounts for queues that carry over to the next analysis period and produces a more accurate representation of delay. This approach, which is called a multiple time-period analysis, is described in the next subsection.

Regardless of analysis period duration, a single-period analysis (i.e., Approach A or B) is typical for planning applications.

Multiple Time-Period Analysis

If the analysis period's demand volume exceeds capacity, then a multiple time-period analysis should be undertaken when the study period includes an initial analysis period with no initial queue and a final analysis period with no residual queue. The initial queue for the second and subsequent periods is equal to the residual queue from the previous period. This approach provides a more accurate estimate of the delay associated with the congestion.

If evaluation of multiple analysis periods is determined to be important, then the performance estimates for each period should be separately reported. In this situation, reporting an average performance for the study period is not encouraged because it may obscure extreme values and suggest acceptable operation when in reality some analysis periods have unacceptable operation.

Performance Measures

Performance measures applicable to the motorized vehicle travel mode include volume-to-capacity ratio, control delay, and queue storage ratio. The queue storage ratio describes the ratio of the back-of-queue size to the available vehicle storage length. The back of queue represents the maximum backward extent of queued vehicles during a typical cycle.

LOS is also considered a performance measure. It is useful for describing intersection performance to elected officials, policy makers, administrators, or the public. LOS is based on control delay.

Limitations of the Methodology

This subsection identifies the known limitations of the motorized vehicle methodology. If one or more of these limitations are believed to have an important influence on the performance of a specific street segment, then the analyst should consider using alternative methods or tools for the evaluation.

The motorized vehicle methodology does not account for the effect of the following conditions on intersection operation:

- Turn bay overflow;
- Multiple advance detectors in the same lane;
- Demand starvation due to a closely spaced upstream intersection;
- Queue spillback into the subject intersection from a downstream intersection;
- Queue spillback from the subject intersection into an upstream intersection;
- Premature phase termination due to short detection length, passage time, or both;
- Right-turn-on-red (RTOR) volume prediction or resulting right-turn delay;
- Turn movements served by more than two exclusive lanes;
- Delay to traffic movements that are not under signal control;
- Through lane (or lanes) added just upstream of the intersection or dropped just downstream of the intersection; and
- Storage of shared-lane left-turning vehicles within the intersection to permit bypass by through vehicles in the same lane.

In addition to the above conditions, the methodology does not directly account for the following controller functions:

- Rest-in-walk mode for actuated and noncoordinated phases,
- Preemption or priority modes,
- Phase overlap (see discussion that follows), and
- Gap reduction or variable initial settings for actuated phases.

Two control strategies that use phase overlap are addressed by the methodology. One strategy is "right-turn overlap with the complementary left-turn phase." This strategy uses the overlap feature to provide a protected signal indication for the right-turn movement that is concurrent with the complementary left-turn phase. Procedures for evaluating this operation are described in Chapter 31, Signalized Intersections: Supplemental. They address the case in which the right-turn movement has an exclusive turn lane.

The second strategy, "left-turn overlap with the opposing through phase," uses the overlap feature to provide the signal indication for permissive left-turn operation that is concurrent with the opposing through phase. This overlap feature is referred to as "Dallas left-turn phasing." It is typically used with flashing yellow operation. Procedures for evaluating this operation are described in Chapter 31.

Lane Groups and Movement Groups

The motorized vehicle methodology uses the concepts of *lane groups* and *movement groups* to describe and evaluate intersection operation. These two group designations are very similar in meaning. In fact, their differences emerge only when a shared lane is present on an approach with two or more lanes. Each designation is defined in the following paragraphs. Guidelines for establishing lane groups and movement groups are described in the subsection titled Computational Steps.

Lane Groups

The motorized vehicle methodology is designed to analyze the performance of designated lanes, groups of lanes, an intersection approach, and the entire intersection. A lane or group of lanes designated for separate analysis is referred to as a *lane group*. In general, a separate lane group is established for (*a*) each lane (or combination of adjacent lanes) that exclusively serves one movement and (*b*) each lane shared by two or more movements. A lane group can include one or more lanes.

The concept of lane groups is useful when a shared lane is present on an approach that has two or more lanes. Several procedures in the methodology require some indication of whether the shared lane serves a mix of vehicles or functions as an exclusive turn lane. This issue cannot be resolved until the proportion of turns in the shared lane has been computed. If the computed proportion of turns in the shared lane equals 1.0 (i.e., 100%), the shared lane is considered to operate as an exclusive turn lane.

Movement Groups

The concept of *movement groups* is established to facilitate data entry to the methodology. In this regard, input data describing intersection traffic are traditionally specific to the movement (e.g., left-turn movement volume); the data are not specific to the lane (e.g., analysts rarely have the volume for a lane shared by left-turning and through vehicles). Thus, most traffic characteristic–related and geometric design–related input data are specific to a movement group.

The basic principle for establishing movement groups is that no traffic movement can be assigned to more than one movement group. Thus, a separate movement group is established for (*a*) each turn movement with one or more exclusive turn lanes and (*b*) the through movement (inclusive of any turn movements that share a lane). A movement group can include one or more lanes.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the motorized vehicle methodology. These data are listed in Exhibit 19-11. The additional data needed for coordinated control are listed in Exhibit 19-12. The second column (labeled Basis) of both Exhibit 19-11 and Exhibit 19-12 indicates whether the input data are needed for each traffic movement, a specific movement group, each signal phase, each intersection approach, or the intersection as a whole. The exhibits also list potential data sources and default values that can be used if local data are not available (*30*).

The data elements listed in Exhibit 19-11 and Exhibit 19-12 do not include variables that are considered to represent calibration factors (e.g., start-up lost time). A calibration factor typically has a relatively narrow range of reasonable values or has a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at relevant points in the presentation of the methodology.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 19-11. These data describe the motorized vehicle traffic stream that travels through the intersection during the study period.

Demand Flow Rate

The demand flow rate for an intersection traffic movement is defined as the count of vehicles arriving at the intersection during the analysis period divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. Demand flow rate represents the flow rate of vehicles *arriving* at the intersection. The "count of vehicles" can be obtained from a variety of sources (e.g., from the field or as a forecast from a planning model).

When measured in the field, the demand flow rate is based on a traffic count taken upstream of the queue associated with the subject intersection. This distinction is important for counts during congested periods because the count of vehicles departing from a congested approach will produce a demand flow rate estimate that is lower than the true rate (i.e., the estimate will equal capacity rather than true demand).

Exhibit 19-11
Required Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
		Traffic Characteristics	
Demand flow rate (veh/h)	M	Field data, past counts	Must be provided
Right-turn-on-red flow rate (veh/h)	A	Field data, past counts	0.0 veh/h
Percentage heavy vehicles (%)	MG	Field data, past counts	3%
Peak hour factor (decimal)	I	Field data, analyst judgment	Hourly data and 0.25-h analysis period: Total entering vol. ≥1,000 veh/h: 0.92 Total entering vol. <1,000 veh/h: 0.90 Otherwise: 1.00
Platoon ratio (decimal)	MG	Field data, analyst judgment	See discussion
Upstream filtering adjustment factor (decimal)	MG	Field data, analyst judgment	1.0
Initial queue (veh)	MG	Field data, analyst judgment	Must be provided
Base saturation flow rate (pc/h/ln)	MG	Field data, analyst judgment	Metro pop. ≥250,000: 1,900 pc/h/ln Otherwise: 1,750 pc/h/ln
Lane utilization adjustment factor (decimal)	MG	Field data, analyst judgment	See discussion
Pedestrian flow rate (p/h)	A	Field data, past counts	Must be provided
Bicycle flow rate (bicycles/h)	A	Field data, past counts	Must be provided
On-street parking maneuver rate (veh/h)	MG	Field data, analyst judgment	See discussion
Local bus stopping rate (buses/h)	A	Field data, analyst judgment	CBD bus stop: 12 buses/h Non-CBD bus stop: 2 buses/h
Unsignalized movement delay (s)	M	Field data	See discussion
		Geometric Design	
Number of lanes (ln)	M	Field data, aerial photo	Must be provided
Average lane width (ft)	MG	Field data, aerial photo	12 ft
Number of receiving lanes (ln)	A	Field data, aerial photo	Must be provided
Turn bay length (ft)	MG	Field data, aerial photo	Must be provided
Presence of on-street parking	MG	Field data, aerial photo	Must be provided
Approach grade (%)	A	Field data	Flat approach: 0% Moderate grade on approach: 3% Steep grade on approach: 6%
		Signal Control	
Type of signal control	I	Field data	Must be provided
Phase sequence	A	Field data	Must be provided
Left-turn operational mode	A	Field data	Must be provided
Dallas left-turn phasing option	A	Field data	Dictated by local use
Passage time (s) (if actuated)	P	Field data	2.0 s (presence detection)
Maximum green (s) (if actuated)	P	Field data	Major-street through movement: 50 s Minor-street through movement: 30 s Left-turn movement: 20 s
Green duration (s) (if pretimed)	P	Field data	Major-street through movement: 50 s Minor-street through movement: 30 s Left-turn movement: 20 s
Minimum green (s)	P	Field data	Major-street through movement: 10 s Minor-street through movement: 8 s Left-turn movement: 6 s
Yellow change + red clearance (s)[a]	P	Field data	4.0 s
Walk (s)	P	Field data	Actuated: 7.0 s Pretimed: green interval minus pedestrian clear
Pedestrian clear (s)	P	Field data	Based on 3.5-ft/s walking speed
Phase recall (if actuated)	P	Field data	No recall
Dual entry (if actuated)	P	Field data	Not enabled (i.e., use single entry)
Simultaneous gap-out (if actuated)	A	Field data	Enabled
Cycle length (if pretimed)	I	Field data	See discussion

Note: Exhibit continues on the next page.

Exhibit 19-11 (cont'd.)
Required Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
		Other Data	
Analysis period duration (h)	I	Set by analyst	0.25 h
Speed limit (mi/h)	A	Field data, road inventory	Must be provided
Stop-line detector length (ft) and detection mode (if actuated)	MG	Field data	40 ft, presence detection mode
Area type (CBD, non-CBD)	I	Analyst judgment	Must be provided

Notes: M = movement: one value for each left-turn, through, and right-turn movement.
A = approach: one value or condition for the intersection approach.
MG = movement group: one value for each turn movement with exclusive turn lanes and one value for the through movement (inclusive of any turn movements in a shared lane).
I = intersection: one value or condition for the intersection.
P = phase: one value or condition for each signal phase.
CBD = central business district; vol. = volume; pop. = population.
[a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.

Exhibit 19-12
Required Additional Input Data, Potential Data Sources, and Default Values for Motorized Vehicle Analysis with Coordinated Signal Control

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
Cycle length (s)	I	Field data	See discussion
Phase splits (s) (if actuated)	P	Field data	See discussion
Offset (s)	I	Field data	Equal to travel time in Phase 2 direction[a]
Offset reference point (s)	I	Field data	End of green for Phase 2[a]
Force mode (if actuated)	I	Field data	Fixed

Notes: I = intersection: one value or condition for the intersection.
P = phase: one value or condition for each signal phase.
[a] Assumes Phase 2 is the reference phase. Substitute 6 if Phase 6 is the reference phase.

If a planning analysis is being conducted in which (*a*) the projected demand flow rate coincides with a 1-h period and (*b*) an analysis of the peak 15-min period is desired, then each movement's hourly demand should be divided by the intersection peak hour factor to predict the flow rate during the peak 15-min period. The peak hour factor should be based on local traffic peaking trends. If a local factor is not available, then the default value in Exhibit 19-11 can be used.

If a multiple time-period analysis is conducted (i.e., Approach C in Exhibit 19-10), then the intersection's demand flow rates should be provided for each analysis period.

The methodology includes a procedure for determining the distribution of flow among the available lanes on an approach with one or more shared lanes. The procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection. This assumption may not hold for situations in which drivers choose a lane so they are prepositioned for a turn at the downstream intersection. Similarly, it may not hold when an auxiliary through lane is present. In either situation, the analyst will need to provide the demand flow rate for each lane on the approach and then combine these rates to define the demand flow rate for each lane group. Additional discussion of this topic is provided in the subsection titled Lane Utilization Adjustment Factor.

The demand flow rate for all signal-controlled movements must be provided. The demand flow rate for all unsignalized movements should be provided. If an unsignalized movement exists but its flow rate is not provided, then this movement will be excluded from the calculation of approach delay and intersection delay.

Right-Turn-on-Red Flow Rate

The RTOR flow rate is defined as the count of vehicles that turn right at the intersection when the controlling signal indication is red, divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h.

It is difficult to predict the RTOR flow rate because it is based on many factors that vary widely from intersection to intersection. These factors include the following:

- Approach lane allocation (shared or exclusive right-turn lane),
- Right-turn flow rate,
- Sight distance available to right-turning drivers,
- Volume-to-capacity ratio for conflicting movements,
- Arrival patterns of right-turning vehicles during the signal cycle,
- Departure patterns of conflicting movements,
- Left-turn signal phasing on the conflicting street, and
- Conflicts with pedestrians.

Given the difficulty of estimating the RTOR flow rate, it should be measured in the field when possible. If the analysis is dealing with future conditions or if the RTOR flow rate is not known from field data, then the RTOR flow rate for each right-turn movement should be assumed to equal 0 veh/h. This assumption is conservative because it yields a slightly larger estimate of delay than may actually be incurred by intersection movements.

Percentage Heavy Vehicles

A heavy vehicle is defined as any vehicle with more than four tires touching the pavement. Local buses that stop within the intersection area are not included in the count of heavy vehicles. The percentage of heavy vehicles represents the count of heavy vehicles that arrive during the analysis period divided by the total vehicle count for the same period. This percentage is provided for each intersection traffic movement; however, one representative value for all movements may be used for a planning analysis.

Intersection Peak Hour Factor

One peak hour factor for the entire intersection is computed with Equation 19-4.

$$PHF = \frac{n_{60}}{4\, n_{15}}$$

Equation 19-4

where

PHF = peak hour factor,

n_{60} = count of vehicles during a 1-h period (veh), and

n_{15} = count of vehicles during the peak 15-min period (veh).

The count used in the denominator of Equation 19-4 must be taken during a 15-min period that occurs within the 1-h period represented by the variable in the numerator. Both variables in this equation represent the total number of vehicles entering the intersection during their respective time period. As such, one peak hour factor is computed for the intersection. This factor is then applied individually to each traffic movement. Values of this factor typically range from 0.80 to 0.95.

The peak hour factor is used primarily for a planning analysis when a forecast hourly volume is provided and an analysis of the peak 15-min period is sought.

The peak hour factor is used primarily for a planning analysis when a forecast hourly volume is provided and an analysis of the peak 15-min period is sought. Normally, the demand flow rate is computed as the count of vehicles arriving during the period divided by the length of the period, expressed as an hourly flow rate, and without the use of a peak hour factor.

If the peak hour factor is used, a single intersectionwide factor should be used rather than movement-specific or approach-specific factors. If individual approaches or movements peak at different times, a series of 15-min analysis periods that encompass the peaking should be considered.

If peak hour factors are used, a single peak hour factor for the entire intersection is generally preferred because it will decrease the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, but in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. In the event individual approaches or movements are known to have substantially different peaking characteristics or peak during different 15-min periods within the hour, a series of 15-min analysis periods that encompass the peaking should be considered instead of a single analysis period using a single peak hour factor for the intersection.

Platoon Ratio

Platoon ratio is used to describe the quality of signal progression for the corresponding movement group. It is computed as the demand flow rate during the green indication divided by the average demand flow rate. Values for the platoon ratio typically range from 0.33 to 2.0. Exhibit 19-13 provides an indication of the quality of progression associated with selected platoon ratio values.

Exhibit 19-13
Relationship Between Arrival Type and Progression Quality

Platoon Ratio	Arrival Type	Progression Quality
0.33	1	Very poor
0.67	2	Unfavorable
1.00	3	Random arrivals
1.33	4	Favorable
1.67	5	Highly favorable
2.00	6	Exceptionally favorable

For protected or protected-permitted left-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the associated turn phase (i.e., the protected period). Hence, the platoon ratio is based on the flow rate during the green indication of the left-turn phase.

For permitted left-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the permitted period. Hence, the platoon ratio is based on the left-turn flow rate during the green indication of the phase providing the permitted operation.

For permitted or protected-permitted right-turn movements operating in an exclusive lane, platoon ratio is used to describe progression quality during the permitted period (even if a protected right-turn operation is provided during the complementary left-turn phase on the cross street). Hence, the platoon ratio is based on the right-turn flow rate during the green indication of the phase providing the permitted operation.

For through movements served by exclusive lanes (no shared lanes on the approach), the platoon ratio for the through movement group is based on the through flow rate during the green indication of the associated phase.

For all movements served by split phasing, the platoon ratio for a movement group is based on its flow rate during the green indication of the common phase.

For intersection approaches with one or more shared lanes, one platoon ratio is computed for the shared movement group on the basis of the flow rate of all shared lanes (plus that of any exclusive through lanes that are also served) during the green indication of the common phase.

The platoon ratio for a movement group can be estimated from field data by using Equation 19-5.

$$R_p = \frac{P}{(g/C)}$$

Equation 19-5

where

R_p = platoon ratio,

P = proportion of vehicles arriving during the green indication (decimal),

g = effective green time (s), and

C = cycle length (s).

P is computed as the count of vehicles that arrive during the green indication divided by the count of vehicles that arrive during the entire signal cycle. It is an average value representing conditions during the analysis period.

Determining Platoon Ratio

If the subject intersection is part of a signal system, then the procedure in Section 3 of Chapter 30, Urban Street Segments: Supplemental, can be used to estimate the arrival flow profile for any approach that is evaluated as part of an urban street segment. The procedure uses the flow profile to compute the proportion of arrivals during the green indication. If this procedure is used, then platoon ratio is not an input for the traffic movements on the subject approach.

If the subject intersection is not part of a signal system and an existing intersection is being evaluated, then it is recommended that analysts use field-measured values for the variables in Equation 19-5 in estimating the platoon ratio.

If the subject intersection is not part of a signal system and the analysis deals with future conditions, or if the variables in Equation 19-5 are not known from field data, then the platoon ratio can be estimated by using guidance provided in the next subsection.

Guidance for Estimating Platoon Ratio

This subsection provides guidance for estimating arrival type. The platoon ratio can then be determined from this arrival type by using Exhibit 19-13.

Exhibit 19-14 provides guidance for selecting the arrival type for through movements. The guidance considers typical signal spacing and the provision of signal coordination. Arrival Type 3 is typically assumed for turn movements.

Exhibit 19-14
Arrival Type Selection Guidelines

Arrival Type	Typical Signal Spacing (ft)	Conditions Under Which Arrival Type Is Likely to Occur
1	≤1,600	Coordinated operation on a two-way street where the subject direction does not receive good progression
2	>1,600–3,200	A less extreme version of Arrival Type 1
3	>3,200	Isolated signals or widely spaced coordinated signals
4	>1,600–3,200	Coordinated operation on a two-way street where the subject direction receives good progression
5	≤1,600	Coordinated operation on a two-way street where the subject direction receives good progression
6	≤800	Coordinated operation on a one-way street in dense networks and central business districts

A description of each arrival type is provided in the following paragraphs to provide additional assistance with the selection of arrival type.

Arrival Type 1 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the red interval. This arrival type is often associated with short segments with very poor progression in the subject direction of travel (and possibly good progression for the other direction).

Arrival Type 2 is characterized by a moderately dense platoon arriving in the middle of the red interval or a dispersed platoon containing 40% to 80% of the movement group volume arriving throughout the red interval. This arrival type is often associated with segments of average length with unfavorable progression in the subject direction of travel.

Arrival Type 3 describes one of two conditions. If the signals bounding the segment are coordinated, then this arrival type is characterized by a platoon containing less than 40% of the movement group volume arriving partly during the red interval and partly during the green interval. If the signals are not coordinated, then this arrival type is characterized by platoons arriving at the subject intersection at different points in time over the course of the analysis period so that arrivals are effectively random.

Arrival Type 4 is characterized by a moderately dense platoon arriving in the middle of the green interval or a dispersed platoon containing 40% to 80% of the movement group volume arriving throughout the green interval. This arrival type is often associated with segments of average length with favorable progression in the subject direction of travel.

Arrival Type 5 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the green interval. This arrival type is often associated with short segments with highly favorable progression in the subject direction of travel and a low-to-moderate number of side street entries.

Arrival Type 6 is characterized by a dense platoon of more than 80% of the movement group volume arriving at the start of the green interval. This arrival type occurs only on very short segments with exceptionally favorable progression in the subject direction of travel and negligible side street entries. It is reserved for routes in dense signal networks, possibly with one-way streets.

Upstream Filtering Adjustment Factor

The upstream filtering adjustment factor I accounts for the effect of an upstream signal on vehicle arrivals to the subject movement group. Specifically, this factor reflects the way an upstream signal changes the variance in the number of arrivals per cycle. The variance decreases with increasing volume-to-capacity ratio, which can reduce cycle failure frequency and resulting delay.

The filtering adjustment factor varies from 0.09 to 1.0. A value of 1.0 is appropriate for an isolated intersection (i.e., one that is 0.6 mi or more from the nearest upstream signalized intersection). A value of less than 1.0 is appropriate for nonisolated intersections. Equation 19-6 is used to compute I for nonisolated intersections.

$$I = 1.0 - 0.91\,X_u^{2.68} \quad \geq 0.090$$

Equation 19-6

where

I = upstream filtering adjustment factor, and

X_u = weighted volume-to-capacity ratio for all upstream movements contributing to the volume in the subject movement group.

The variable X_u is computed as the weighted volume-to-capacity ratio of all upstream movements contributing to the volume in the subject movement group. This ratio is computed as a weighted average with the volume-to-capacity ratio of each contributing upstream movement weighted by its discharge volume. For planning and design analyses, X_u can be approximated as the volume-to-capacity ratio of the contributing through movement at the upstream signalized intersection. The value of X_u used in Equation 19-6 cannot exceed 1.0.

Initial Queue

The initial queue represents the queue present at the start of the subject analysis period for the subject movement group. This queue is created when oversaturation is sustained for an extended time. The initial queue can be estimated by monitoring queue count continuously during each of the three consecutive cycles that occur just before the start of the analysis period. The smallest count observed during each cycle is recorded. The initial queue estimate equals the average of the three counts. The initial queue estimate should not include vehicles in the queue due to random, cycle-by-cycle fluctuations.

Initial queue has a significant effect on delay and can vary widely among intersections and traffic movements. If it is not possible to obtain an initial queue estimate, then the analysis period should be established so the previous period is known to have demand less than capacity and no residual queue.

Base Saturation Flow Rate

The saturation flow rate represents the maximum rate of flow for a traffic lane as measured at the stop line during the green indication. The base saturation flow rate represents the saturation flow rate for a traffic lane that is 12 ft wide and has no heavy vehicles, a flat grade, no parking, no buses that stop at the intersection, even lane utilization, and no turning vehicles. Typically, one base rate expressed in passenger cars per hour per lane is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. Chapter 31 describes a field measurement technique for quantifying the local base saturation flow rate.

Lane Utilization Adjustment Factor

The lane utilization adjustment factor accounts for the unequal distribution of traffic among the lanes in those movement groups with more than one exclusive lane. This factor provides an adjustment to the base saturation flow rate to account for uneven use of the lanes. It is not used unless a movement group has more than exclusive lane. The lane utilization adjustment factor is calculated with Equation 19-7.

Equation 19-7

$$f_{LU} = \frac{v_g}{N_e\, v_{g1}}$$

where

f_{LU} = adjustment factor for lane utilization,

v_g = demand flow rate for movement group (veh/h),

v_{g1} = demand flow rate in the single exclusive lane with the highest flow rate of all exclusive lanes in movement group (veh/h/ln), and

N_e = number of exclusive lanes in movement group (ln).

Lane flow rates measured in the field can be used with Equation 19-7 to establish local default values of the lane utilization adjustment factor.

A lane utilization factor of 1.0 is used when a movement group has only one lane or a uniform traffic distribution can be assumed across all exclusive lanes in the movement group. Values less than 1.0 apply when traffic is not uniformly distributed. As demand approaches capacity, the lane utilization factor is often closer to 1.0 because drivers have less opportunity to select their lane.

At some intersections, drivers may choose one through lane over another lane in anticipation of a turn downstream. When this type of prepositioning occurs, a more accurate evaluation will be obtained when the actual flow rate for each approach lane is measured in the field and provided as an input to the methodology (in which case a lane utilization factor of 1.0 is used).

Some intersections have an auxiliary through lane (i.e., a through-lane addition on the approach to an intersection combined with a through-lane drop exiting the intersection). This type of lane can be underutilized if it is relatively short. When it is present on an approach, a more accurate evaluation will be obtained when the actual flow rate for each approach lane is provided as input to the methodology. A procedure for estimating approach lane volumes for this situation is provided in NCHRP Report 707 (*31*).

Default Value. The default lane utilization factors described in this subsection apply to situations in which drivers randomly choose among the exclusive-use lanes on the intersection approach. The factors do not apply to special conditions (such as short lane drops or a downstream freeway on-ramp) that might cause drivers intentionally to choose their lane position on the basis of an anticipated downstream maneuver. Exhibit 19-15 provides default lane utilization adjustment factors for different movement groups and numbers of lanes.

Exhibit 19-15
Default Lane Utilization Adjustment Factors

Movement Group	No. of Lanes in Movement Group (ln)	Traffic in Most Heavily Traveled Lane (%)	Lane Utilization Adjustment Factor f_{LU}
Exclusive through	1	100.0	1.000
	2	52.5	0.952
	3[a]	36.7	0.908
Exclusive left turn	1	100.0	1.000
	2[a]	51.5	0.971
Exclusive right turn	1	100.0	1.000
	2[a]	56.5	0.885

Note: [a] If a movement group has more lanes than shown in this exhibit, it is recommended that field surveys be conducted or the smallest f_{LU} value shown for that type of movement group be used.

As demand approaches capacity, the analyst may use lane utilization factors that are closer to 1.0 than those offered in Exhibit 19-15. This refinement to the factor value recognizes that a high volume-to-capacity ratio is associated with a more uniform use of the available lanes because of reduced opportunity for drivers to select their lane freely.

Pedestrian Flow Rate

The pedestrian flow rate is based on the count of pedestrians traveling in the crosswalk that is crossed by vehicles turning right from the subject approach during the analysis period. For example, the pedestrian flow rate for the westbound approach describes the pedestrian flow in the crosswalk on the north leg. A separate count is taken for each direction of travel in the crosswalk. Each count is divided by the analysis period duration to yield a directional hourly flow rate. These rates are then added to obtain the pedestrian flow rate.

Bicycle Flow Rate

The bicycle flow rate is based on the count of bicycles whose travel path is crossed by vehicles turning right from the subject approach during the analysis period. These bicycles may travel on the shoulder or in a bike lane. Any bicycle traffic operating in the right lane with motorized vehicle traffic should not be included in this count. This interaction is not modeled by the methodology. The count is divided by the analysis period duration to yield an hourly flow rate.

On-Street Parking Maneuver Rate

The parking maneuver rate represents the count of *influential* parking maneuvers that occur on an intersection leg as measured during the analysis period. An influential maneuver occurs directly adjacent to a movement group, within a zone that extends from the stop line to a point 250 ft upstream of it. A maneuver occurs when a vehicle enters or exits a parking stall. If more than 180 maneuvers/h exist, then a practical limit of 180 should be used. On a two-way leg, maneuvers are counted for just the right side of the leg. On a one-way leg, maneuvers are separately counted for each side of the leg. The count is divided by the analysis period duration to yield an hourly flow rate.

Exhibit 19-16 gives default values for the parking maneuver rate on an intersection approach with on-street parking. It is estimated for a distance of 250 ft back from the stop line. The calculations assume 25 ft per parking space and 80% occupancy. Each turnover (one car leaving and one car arriving) generates two parking maneuvers.

Exhibit 19-16
Default Parking Maneuver Rate

Street Type	No. of Parking Spaces in 250 ft	Parking Time Limit (h)	Turnover Rate (veh/h)	Maneuver Rate (maneuvers/h)
Two-way	10	1	1.0	16
		2	0.5	8
One-way	20	1	1.0	32
		2	0.5	16

Local Bus Stopping Rate

The bus stopping rate represents the number of local buses that stop and block traffic flow in a movement group within 250 ft of the stop line (upstream or downstream) as measured during the analysis period. A *local bus* is a bus that stops to discharge or pick up passengers at a bus stop. The stop can be on the near side or the far side of the intersection. If more than 250 buses/h exist, then a practical limit of 250 should be used. The count is divided by the analysis period duration to yield an hourly flow rate.

Unsignalized Movement Delay

The delay for unsignalized movements at the intersection should be provided as input to the methodology whenever these movements exist at the intersection. If provided, they can be included in the calculation of approach delay and intersection delay.

The delay will need to be estimated by means external to the methodology. These external means may include direct field measurement, observation of similar conditions, special application of procedures in other HCM chapters, and simulation. The level of effort expended for such estimation should be commensurate with the relevance the unsignalized delay has to the overall analysis. For example, high-volume or high-delay movements should be estimated carefully. Free-flow right turns can be assumed to have zero delay.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 19-11. These data describe the geometric elements of the intersection that influence traffic operation.

Number of Lanes

The number of lanes represents the count of lanes provided for each intersection traffic movement. For a turn movement, this count represents the lanes reserved for the exclusive use of turning vehicles. Turn-movement lanes include turn lanes that extend backward for the length of the segment and lanes in a turn bay. Lanes that are shared by two or more movements are included in the count of through lanes and are described as *shared lanes*. If no exclusive turn lanes are provided, then the turn movement is indicated to have zero lanes.

The number of lanes on an approach depends on approach volume and signal timing. A single exclusive left-turn lane is often provided when the left-turn volume ranges between 100 and 300 veh/h. Similarly, a dual exclusive left-turn lane is often provided when the left-turn volume exceeds 300 veh/h. An exclusive right-turn lane is often provided when the right-turn volume exceeds 300 veh/h and the adjacent through volume exceeds 300 veh/h/ln.

Average Lane Width

The average lane width represents the average width of the lanes represented in a movement group. The minimum average lane width is 8 ft. Standard lane widths are 12 ft. Lane widths greater than 16 ft can be included; however, the analyst should consider whether the wide lane actually operates as two narrow lanes. The analysis should reflect the way in which the lane width is actually used or expected to be used.

Number of Receiving Lanes

The number of receiving lanes represents the count of lanes departing the intersection. This number should be separately determined for each left-turn and right-turn movement. Experience indicates proper turning cannot be executed at some intersections because a receiving lane is frequently blocked by double-parked vehicles. For this reason, the number of receiving lanes should be determined from field observation when possible.

Turn Bay Length

Turn bay length represents the length of the bay for which the lanes have full width and in which queued vehicles can be stored. Bay length is measured parallel to the roadway centerline. If there are multiple lanes in the bay and they have different lengths, then the length entered should be an average value.

If a two-way left-turn lane is provided for left-turn vehicle storage and adjacent access points exist, then the bay length entered should represent the "effective" storage length available to the left-turn movement. The determination of effective length is based on consideration of the adjacent access points and the associated left-turning vehicles that can be stored in the two-way left-turn lane.

Presence of On-Street Parking

The input for presence of on-street parking indicates whether on-street parking is allowed along the curb line adjacent to a movement group and within 250 ft upstream of the stop line during the analysis period. On a two-way street, the presence of parking is noted for just the right side of the street. On a one-way street, the presence of on-street parking is separately noted for each side of the street.

Approach Grade

Approach grade defines the average grade along the approach as measured from the stop line to a point 100 ft upstream of the stop line along a line parallel to the direction of travel. An uphill condition has a positive grade, and a downhill condition has a negative grade.

Signal Control Data

This subsection describes the signal control data listed in Exhibit 19-11 and Exhibit 19-12. They are specific to an actuated signal controller that is operated in a pretimed, semiactuated, fully actuated, or coordinated-actuated manner.

Type of Signal Control

The intersection signal control is an input to the methodology. It can be pretimed control or actuated control. Pretimed control can be described as coordinated (or coordinated-pretimed) if the intersection is part of a signal system. Actuated control can be described as fully actuated, semiactuated, or coordinated-actuated.

Settings used for coordinated-actuated control are described later in this subsection. They are used in the motorized vehicle methodology in Chapter 18.

The motorized vehicle methodology is based on the controller functions defined in the National Transportation Communications for ITS Protocol Standard 1202 (*28*). It is incumbent on the analyst to become familiar with these functions and adapt them, if needed, to the functionality of the controller used at the subject intersection. Section 2 provides additional information about traffic signal controller operation.

Phase Sequence

In broad context, phase sequence describes the order of service provided to each traffic movement. This definition is narrowed here to limit phase sequence to the order in which the left-turn movements are served relative to the through movements. The sequence options addressed in the methodology include no left-turn phase, leading left-turn phase, lagging left-turn phase, and split phasing.

Left-Turn Operational Mode

The left-turn operational mode describes how the left-turn movement is served by the controller. It can be described as permitted, protected, or protected-permitted.

Dallas Left-Turn Phasing Option

The Dallas left-turn phasing option allows the left-turn movements to operate in the protected-permitted mode without causing a "yellow trap" safety concern. It effectively ties the left turn's permitted-period signal indication to the opposing through movement signal indication. This phasing option is also used with a flashing yellow arrow left-turn signal display.

Passage Time: Actuated Control

Passage time is the maximum amount of time one vehicle actuation can extend the green interval while green is displayed. It is input for each actuated signal phase. It is also referred to as vehicle interval, extension interval, extension, or unit extension.

Passage time values are typically based on detection zone length, detection zone location (relative to the stop line), number of lanes served by the phase, vehicle length, and vehicle speed. Longer passage times are often used with shorter detection zones, greater distance between the zone and stop line, fewer lanes, smaller vehicles, and higher speeds.

The objective in determining the passage time value is to make it large enough to ensure all queued vehicles are served but not so large that it extends for traffic arriving randomly after the queue has cleared. On high-speed approaches, this objective is broadened to include not making the passage time so long that the phase frequently extends to its maximum setting (i.e., max-out) so that safe phase termination is compromised.

Maximum Green: Actuated Control

The maximum green setting defines the maximum amount of time a green signal indication can be displayed in the presence of conflicting demand. Typical maximum green values for left-turn phases range from 15 to 30 s. Typical values for through phases serving the minor-street approach range from 20 to 40 s, and values for through phases serving the major-street approach range from 30 to 60 s.

For an analysis of coordinated-actuated operation, the maximum green is disabled through the inhibit mode, and the phase splits are used to determine the maximum length of the actuated phases.

Green Duration: Pretimed Control

For an analysis of pretimed operation, the green interval duration is an input to the methodology. Typical values are similar to those noted above for the maximum green setting. A procedure for estimating pretimed green interval duration is described in Section 2 of Chapter 31, Signalized Intersections: Supplemental.

Minimum Green

The minimum green setting represents the least amount of time a green signal indication is displayed when a signal phase is activated. Its duration is based on consideration of driver reaction time, queue size, and driver expectancy. Minimum green typically ranges from 4 to 15 s, with shorter values in this range used for phases serving turn movements and lower-volume through movements. For intersections without pedestrian push buttons, the minimum green setting may also need to be long enough to allow time for pedestrians to react to the signal indication and cross the street.

Yellow Change and Red Clearance

The yellow change and red clearance settings are input for each signal phase. The yellow change interval is intended to alert a driver to the impending presentation of a red indication. It ranges from 3 to 6 s, with longer values in this range used with phases serving high-speed movements.

The red clearance interval can be used to allow a brief time to elapse after the yellow indication, during which the signal heads associated with the ending phase and all conflicting phases display a red indication. If used, the red clearance interval is typically 1 or 2 s.

Walk

The walk interval is intended to give pedestrians adequate time to perceive the walk indication and depart the curb before the pedestrian clear interval begins.

For an actuated or noncoordinated phase, the walk interval is typically set at the minimum value needed for pedestrian perception and curb departure. Many agencies consider this value to be 7 s; however, some agencies use as little as 4 s. Longer walk durations should be considered in school zones and areas with large numbers of elderly pedestrians. The methodology assumes the rest-in-walk mode is not enabled for actuated phases and noncoordinated phases.

For a pretimed phase, the walk interval is often set at a value equal to the green interval duration needed for vehicle service less the pedestrian clear setting (provided the resulting interval exceeds the minimum time needed for pedestrian perception and curb departure).

For a coordinated phase, the controller is sometimes set to use a coordination mode that extends the walk interval for most of the green interval duration. This functionality is not explicitly modeled in the motorized vehicle methodology, but it can be approximated by setting the walk interval to a value equal to the phase split minus the sum of the pedestrian clear, yellow change, and red clearance intervals.

If the walk and pedestrian clear settings are provided for a phase, then it is assumed a pedestrian signal head is also provided. If these settings are not used, then it is assumed any pedestrian accommodation needed is provided in the minimum green setting.

Pedestrian Clear

The pedestrian clear interval (also referred to as the pedestrian change interval) is intended to provide time for pedestrians who depart the curb during the WALK indication to reach the opposite curb (or the median). Some agencies set the pedestrian clear equal to the "crossing time," where crossing time equals the curb-to-curb crossing distance divided by the pedestrian walking speed of 3.5 ft/s. Other agencies set the pedestrian clear equal to the crossing time less the vehicle change period (i.e., the combined yellow change and red clearance intervals). This choice depends on agency policy and practice. A flashing DON'T WALK indication is displayed during this interval.

Phase Recall: Actuated Control

If used, recall causes the controller to place a call for a specified phase each time the controller is servicing a conflicting phase. It is input for each signal phase. Three types of recalls are modeled in the motorized vehicle methodology: minimum recall, maximum recall, and pedestrian recall.

Invoking minimum recall causes the controller to place a continuous call for vehicle service on the phase and then service the phase until its minimum green interval times out. The phase can be extended if actuations are received.

Invoking maximum recall causes the controller to place a continuous call for vehicle service on the phase. It results in presentation of the green indication for its maximum duration every cycle. Using maximum recall on all phases yields an equivalent pretimed operation.

Invoking pedestrian recall causes the controller to place a continuous call for pedestrian service on the phase and then service the phase for at least an amount of time equal to its walk and pedestrian clear intervals (longer if vehicle detections are received). Pedestrian recall is used for phases that have a high probability of pedestrian demand every cycle and no pedestrian detection.

Dual Entry: Actuated Control

The entry mode is used in dual-ring operation to specify whether a phase is to be activated (green) even though it has not received a call for service. Two entry modes are possible: dual entry and single entry. This mode is input for each actuated signal phase.

A phase operating in dual entry is available to be called by the controller, even if no actuations have been received for this phase. A phase operating in single entry will be called only if actuations have been received.

During the timing of a cycle, a point is reached at which the next phase (or phases) to be timed is on the other side of a barrier. At this point, the controller will check the phases in each ring and determine which phase to activate. If a call does not exist in a ring, the controller will activate a phase designated as dual entry in that ring. If two phases are designated as dual entry in the ring, then the first phase to occur in the phase sequence is activated.

Simultaneous Gap-Out: Actuated Control

The simultaneous gap-out mode affects the way actuated phases are terminated before the barrier can be crossed to serve a conflicting call. This mode can be enabled or disabled. It is a phase-specific setting; however, it is typically set the same for all phases that serve the same street. This mode is input for each actuated signal phase.

Simultaneous gap-out dictates controller operation when a barrier must be crossed to serve the next call and one phase is active in each ring. If simultaneous gap-out is enabled, both phases must reach a point of being committed to terminate (via gap-out, max-out, or force-off) at the same time. If one phase is able to terminate because it has gapped out, but the other phase is not able to terminate, then the gapped-out phase will reset its extension timer and restart the process of timing down to gap-out.

If the simultaneous gap-out feature is disabled, then each phase can reach a point of termination independently. In this situation, the first phase to commit to termination maintains its active status while waiting for the other phase to commit to termination. Regardless of which mode is in effect, the barrier is not crossed until both phases are committed to terminate.

Cycle Length: Coordinated-Actuated or Pretimed Control

Cycle length is the time elapsed between the endings of two sequential presentations of a coordinated-phase green interval. A cycle length is needed for pretimed control and for coordinated-actuated control.

Default Value. The cycle length used for a coordinated signal system often represents a compromise value based on intersection capacity, queue size, phase sequence, segment length, speed, and progression quality. Consideration of these factors leads to the default cycle lengths shown in Exhibit 19-17.

Exhibit 19-17
Default System Cycle Length

	Cycle Length by Street Class and Left-Turn Phasing (s)[b]					
	Major Arterial Street			Minor Arterial Street or Grid Network		
Average Segment Length (ft)[a]	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets	No Left-Turn Phases	Left-Turn Phases on One Street	Left-Turn Phases on Both Streets
1,300	90	120	150	60	80	120
2,600	90	120	150	100	100	120
3,900	110	120	150			

Notes: [a] Average length based on all street segments in the signal system.
[b] Selected left-turn phasing column should describe the phase sequence at the high-volume intersections in the system.

Phase Splits: Coordinated-Actuated Control

Each noncoordinated phase is provided a "split" time that represents the sum of the green, yellow change, and red clearance intervals for the phase. The rationale for determining the green interval duration varies among agencies; however, it is often related to the "optimum" pretimed green interval duration. Section 2 of Chapter 31 provides a procedure for determining pretimed phase duration.

If the phase splits are not known, they can be estimated by using the planning-level analysis application described in Chapter 31. The method can be used to estimate the effective green time for each phase on the basis of the established system cycle length. The phase split D_p is then computed by adding 4 s of lost time to the estimated effective green time (i.e., $D_p = g + 4.0$).

Offset and Offset Reference Point: Coordinated Control

The reference phase is specified to be one of the two coordinated phases (i.e., Phase 2 or 6). The offset entered in the controller represents the time the reference phase begins (or ends) relative to the system master time zero. The offset must be specified as being referenced to the beginning (or the end) of the green interval of the reference phase. The offset reference point is typically the same at all intersections in a given signal system.

Force Mode: Coordinated-Actuated Control

The force mode is a controller-specific setting. It is set to "fixed" or "floating." The controller calculates the phase force-off point for each noncoordinated phase on the basis of the force mode and the phase splits. When set to the fixed mode, each noncoordinated phase has its force-off point set at a fixed time in the cycle relative to time zero on the system master. This operation allows unused split time to revert to the following phase. When set to the floating mode, each noncoordinated phase has its force-off point set at the split time after the phase first becomes active. This operation allows unused split time to revert to the coordinated phase (referred to as an "early return to green").

Other Data

This subsection describes the data listed in Exhibit 19-11 that are categorized as "other" data.

Analysis Period Duration

The analysis period is the time interval considered for the performance evaluation. Its duration ranges from 15 min to 1 h, with longer durations in this range sometimes used for planning analyses. In general, the analyst should use caution in interpreting the results from an analysis period of 1 h or more because the adverse impact of short peaks in traffic demand may not be detected.

Any 15-min period of interest can be evaluated with the methodology; however, a complete evaluation should always include an analysis of conditions during the 15-min period that experiences the highest traffic demand during a 24-h period.

Operational analyses require a 15-min analysis period. This duration will accurately capture the adverse effects of demand peaks.

Most planning analyses use a 15-min analysis period. However, a 1-h analysis period can be used, if appropriate.

Speed Limit

The methodology is based on the assumption that the posted speed limit is (*a*) consistent with posted speed limits found on other streets in the vicinity of the subject intersection and (*b*) consistent with agency policy regarding specification of speed limits. If it is known the posted speed limit does not satisfy these assumptions, then the speed limit value that is input to the methodology should be adjusted so it is consistent with the assumptions.

Stop-Line Detector Length and Detection Mode: Actuated Control

The stop-line detector length represents the length of the detection zone used to extend the green indication. This detection zone is typically located near the stop line and may have a length of 40 ft or more. However, it can be located some distance upstream of the stop line and may be as short as 6 ft. The latter configuration typically requires a long minimum green or use of the controller's variable initial setting.

If a video-image vehicle detection system is used to provide stop-line detection, then the input length should reflect the physical length of roadway that is monitored by the video detection zone plus a length of 5 to 10 ft to account for the projection of the vehicle image into the plane of the pavement (with larger values in this range used for wider intersections).

Detection mode influences the duration of the actuation submitted to the controller by the detection unit. One of two modes can be used: presence or pulse. Presence mode is typically the default mode. It tends to provide more reliable intersection operation than pulse mode.

In the presence mode, the actuation starts with the vehicle arriving in the detection zone and ends with the vehicle leaving the detection zone. Thus, the time duration of the actuation depends on vehicle length, detection zone length, and vehicle speed.

The presence mode is typically used with long detection zones located at the stop line. The combination typically results in the need for a small passage-time value. This characteristic is desirable because it tends to result in efficient queue service.

In the pulse mode, the actuation starts and ends with the vehicle arriving at the detector (actually, the actuation is a short "on" pulse of 0.10 to 0.15 s). This mode is not used as often as presence mode for intersection control.

Area Type

The area type input indicates whether an intersection is in a central business district (CBD) type of environment. An intersection is considered to be in a CBD, or a similar type of area, when its characteristics include narrow street rights-of-way, frequent parking maneuvers, vehicle blockages, taxi and bus activity, small-radius turns, limited use of exclusive turn lanes, high pedestrian activity, dense population, and midblock curb cuts. The average saturation headway at intersections in areas with CBD-like characteristics is significantly longer than at intersections in areas that are less constrained and less visually intense.

OVERVIEW OF THE METHODOLOGY

This subsection provides an overview of the methodology for evaluating the performance of the signalized intersection in terms of its service to motorized vehicles. The methodology is computationally intense and is most efficiently implemented using software. The intensity stems partly from the need to model traffic-actuated signal operation.

A planning-level analysis application for evaluating intersection performance is provided in Chapter 31, Signalized Intersections: Supplemental. This method is not computationally intense and can be applied by using hand calculations.

The objective of this overview is to introduce the analyst to the calculation process and discuss the key analytic procedures. This objective is achieved by focusing the discussion on lane groups that serve one traffic movement with pretimed control and for which there are no permitted or protected-permitted left-turn movements. Details on evaluation of actuated control, shared-lane lane groups, and intersections with permitted or protected-permitted left-turn operation are provided in Chapter 31.

The computational engine developed by the Transportation Research Board Committee on Highway Capacity and Quality of Service represents the most detailed description of this methodology. Additional information about this engine is provided in Chapter 31.

Framework

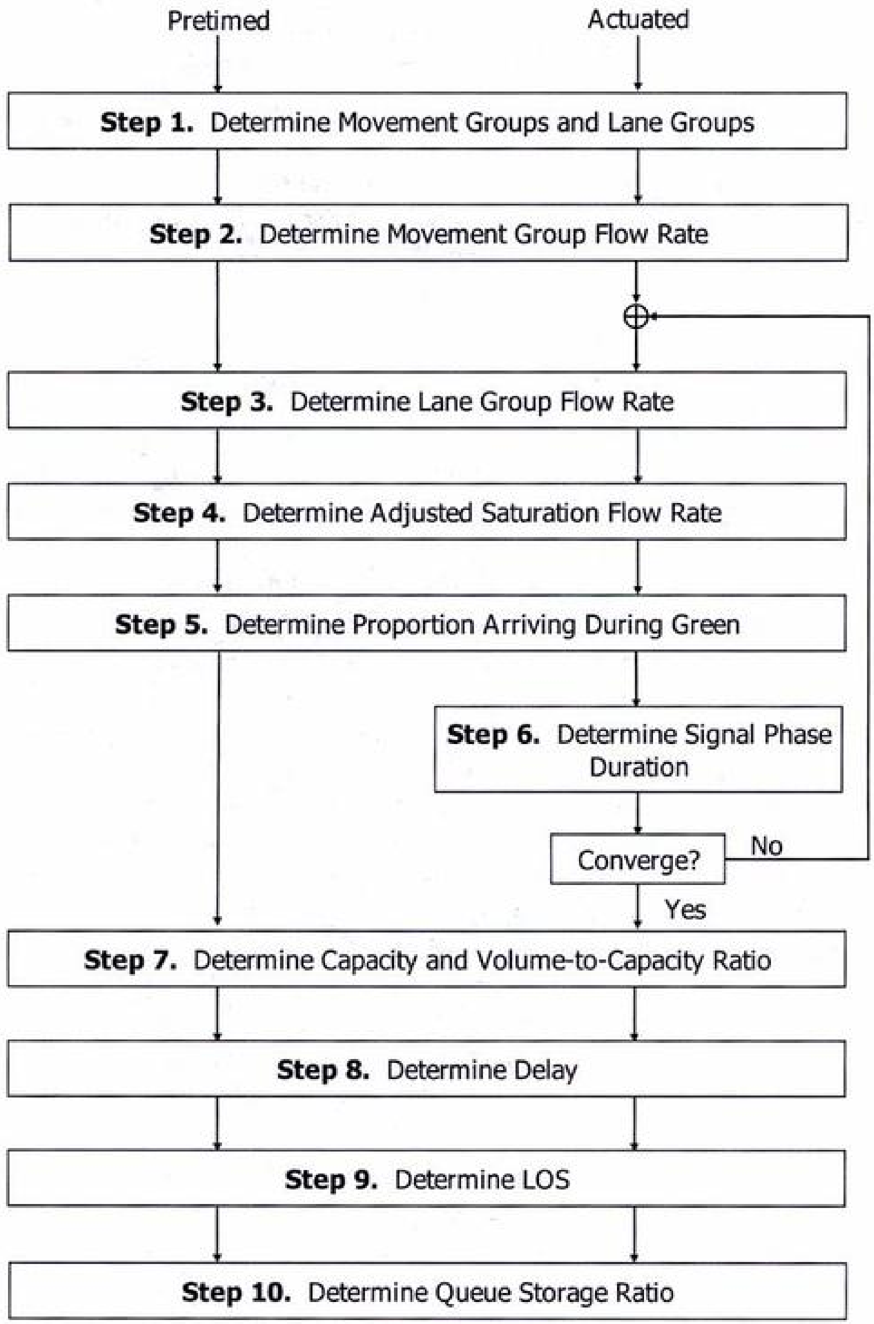
Exhibit 19-18 illustrates the calculation framework of the motorized vehicle methodology. It identifies the sequence of calculations needed to estimate selected performance measures. The calculation process is shown to flow from top to bottom in the exhibit. These calculations are described more fully in the next section.

The methodology adopts the same principles of critical movement analysis used in prior editions of the HCM. The first step of the methodology is used to determine the lane groups associated with each intersection approach. These lane groups represent the basic unit of analysis. Each lane group is separately evaluated, and the results are aggregated to the approach and intersection levels. The second and third steps are used to determine how the left-turn, through, and right-turn drivers on each intersection approach distribute themselves among the lane groups. The fourth step is used to predict the saturation flow rate for each lane group based on prevailing conditions. The fifth step is used to quantify the effect of upstream signals on the arrival flow rate for each lane group. If a phase is actuated, the sixth step is needed to estimate the average duration of this phase. In the seventh step, lane group capacity is evaluated in terms of the ratio of flow rate to capacity. This ratio is used in Step 8 to estimate the control delay for each lane group. This estimated control delay is used in Step 9 to estimate the LOS for each lane group, approach, and intersection. The tenth step can be optionally used to estimate lane group queue length and storage ratio.

For actuated control, the methodology is shown to be iterative within Steps 3 to 6, with convergence achieved when the predicted phase duration and capacity from successive iterations are effectively in agreement. Before the first iteration, an initial rough estimate of the phase duration is made to support the calculations in Steps 3 to 5. A revised estimate of phase duration is produced in Step 6. The revised estimate is compared with the previous estimate and, if they are not in agreement, the process is repeated until convergence is achieved. Several iterations are typically needed.

Although not shown in Exhibit 19-18, Step 3 includes an iterative procedure that is used when one or more lane groups have a shared lane. This procedure allocates the through volume among the available shared and exclusive through lanes to determine the lane group volume assignment that produces the lowest service time. It is implemented for both pretimed and actuated control.

Exhibit 19-18
Motorized Vehicle Methodology for Signalized Intersections



COMPUTATIONAL STEPS

Step 1: Determine Movement Groups and Lane Groups

The movement groups and lane groups are established during this step. They are established separately for each intersection approach. Rules for establishing these groups are described in the subsequent paragraphs. Exhibit 19-19 shows some common movement groups and lane groups. A discussion of the need for, and difference between, movement groups and lane groups is provided in a previous subsection titled Lane Groups and Movement Groups.

Exhibit 19-19
Typical Movement Groups and Lane Groups

Number of Lanes	Movements by Lanes	Movement Groups (MG)	Lane Groups (LG)
1	Left, through, and right:	MG 1:	LG 1:
2	Exclusive left: Through and right:	MG 1: MG 2:	LG 1: LG 2:
2	Left and through: Through and right:	MG 1:	LG 1: LG 2:
3	Exclusive left: Exclusive left: Through: Through: Through and right:	MG 1: MG 2:	LG 1: LG 2: LG 3:

Determine Movement Groups

The following rules are used to determine movement groups for an intersection approach:

- A turn movement served by one or more exclusive lanes and no shared lanes should be designated as a movement group.
- Any lanes not assigned to a group by the previous rule should be combined into one movement group.

These rules result in the designation of one to three movement groups for each approach. A movement group can include one or more lanes.

Determine Lane Groups

A lane group can include one or more lanes. The following rules are used to determine lane groups for an intersection approach:

- An exclusive left-turn lane or lanes should be designated as a separate lane group. The same is true of an exclusive right-turn lane.
- Any shared lane should be designated as a separate lane group.
- Any lanes that are not exclusive turn lanes or shared lanes should be combined into one lane group.

These rules result in the designation of one or more of the following lane group possibilities for an intersection approach:

- Exclusive left-turn lane (or lanes),
- Exclusive through lane (or lanes),
- Exclusive right-turn lane (or lanes),
- Shared left-turn and through lane,
- Shared left-turn and right-turn lane,
- Shared right-turn and through lane, and
- Shared left-turn, through, and right-turn lane.

Step 2: Determine Movement Group Flow Rate

The flow rate for each movement group is determined in this step. If a turn movement is served by one or more exclusive lanes and no shared lanes, then that movement's flow rate is assigned to a movement group for the exclusive lanes. Any of the approach flow that is yet to be assigned to a movement group (after application of the guidance in the previous sentence) is assigned to one movement group.

The RTOR flow rate is subtracted from the right-turn flow rate, regardless of whether the right turn occurs from a shared or an exclusive lane. The reduced right-turn volume is used to compute capacity and LOS in subsequent steps.

Step 3: Determine Lane Group Flow Rate

The lane group flow rate is determined in this step. If there are no shared lanes on the intersection approach or the approach has only one lane, there is a one-to-one correspondence between lane groups and movement groups. In this situation, the lane group flow rate equals the movement group flow rate.

If there are one or more shared lanes on the approach and two or more lanes, then the lane group flow rate is computed by the procedure described in Section 2 of Chapter 31 in the subsection titled Lane Group Flow Rate on Multiple-Lane Approaches. This procedure is based on an assumed desire by drivers to choose the lane that minimizes their service time at the intersection, where the lane volume-to-saturation flow ratio is used to estimate relative differences in this time among lanes. This assumption may not always hold for situations in which drivers choose a lane so they are prepositioned for a turn at the downstream intersection. Similarly, it may not hold when an auxiliary through lane is present. In either situation, the analyst needs to provide the flow rate for each lane on the approach and then combine these rates to define explicitly the flow rate for each lane group.

Step 4: Determine Adjusted Saturation Flow Rate

The adjusted saturation flow rate for each lane of each lane group is computed in this step. The base saturation flow rate provided as an input variable is used in this computation.

The computed saturation flow rate is referred to as the adjusted saturation flow rate because it reflects the application of various factors that adjust the base saturation flow rate to the specific conditions present on the subject intersection approach.

The procedure described in this step applies to lane groups that consist of an exclusive lane (or lanes) operating in a pretimed protected mode and without pedestrian or bicycle interaction. When these conditions do not hold, the supplemental procedures described in Sections 2 and 3 of Chapter 31 should be combined with the procedures in this step to compute the adjusted saturation flow rate.

Equation 19-8 is used to compute the adjusted saturation flow rate per lane for the subject lane group.

Equation 19-8

$$s = s_o\, f_w\, f_{HVg}\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{RT}\, f_{Lpb}\, f_{Rpb}\, f_{wz}\, f_{ms}\, f_{sp}$$

where

s = adjusted saturation flow rate (veh/h/ln),

s_o = base saturation flow rate (pc/h/ln),

f_w = adjustment factor for lane width,

f_{HVg} = adjustment factor for heavy vehicles and grade,

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group,

f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area,

f_a = adjustment factor for area type,

f_{LU} = adjustment factor for lane utilization,

f_{LT} = adjustment factor for left-turn vehicle presence in a lane group,

f_{RT} = adjustment factor for right-turn vehicle presence in a lane group,

f_{Lpb} = pedestrian adjustment factor for left-turn groups,

f_{Rpb} = pedestrian–bicycle adjustment factor for right-turn groups,

f_{wz} = adjustment factor for work zone presence at the intersection,

f_{ms} = adjustment factor for downstream lane blockage, and

f_{sp} = adjustment factor for sustained spillback.

The adjustment factors in the list above are described in the following subsections.

Base Saturation Flow Rate

Computations begin with selection of a base saturation flow rate. This base rate represents the expected average flow rate for a through-traffic lane having geometric and traffic conditions that correspond to a value of 1.0 for each adjustment factor. Typically, one base rate is selected to represent all signalized intersections in the jurisdiction (or area) within which the subject intersection is located. Default values for this rate are provided in Exhibit 19-11.

Adjustment for Lane Width

The lane width adjustment factor f_w accounts for the negative impact of narrow lanes on saturation flow rate and allows for an increased flow rate on wide lanes. Values of this factor are listed in Exhibit 19-20.

Exhibit 19-20
Lane Width Adjustment Factor

Average Lane Width (ft)	Adjustment Factor f_w
<10.0[a]	0.96
≥10.0–12.9	1.00
>12.9	1.04

Note: [a] Factors apply to average lane widths of 8.0 ft or more.

Standard lanes are 12 ft wide. The lane width factor may be used with caution for lane widths greater than 16 ft, or an analysis with two narrow lanes may be conducted. Use of two narrow lanes will always result in a higher saturation flow rate than a single wide lane, but, in either case, the analysis should reflect the way the width is actually used or expected to be used. In no case should this factor be used to estimate the saturation flow rate of a lane group with an average lane width that is less than 8.0 ft.

Adjustment for Heavy Vehicles and Grade

The heavy-vehicle and grade adjustment factor f_{HVg} accounts for the combined effect of heavy vehicle and approach grade on saturation flow rate. The heavy-vehicle component of this factor accounts for the additional space occupied by heavy vehicles and for the difference in their operating capabilities compared with passenger cars. The grade component accounts for the effects of approach grade on vehicle performance. An uphill grade has a positive value and a downhill grade has a negative value.

If the grade is negative (i.e., downhill), then the factor is computed with Equation 19-9.

Equation 19-9

$$f_{HVg} = \frac{100 - 0.79\,P_{HV} - 2.07\,P_g}{100}$$

If the grade is not negative (i.e., level or uphill), then the factor is computed with Equation 19-10.

Equation 19-10

$$f_{HVg} = \frac{100 - 0.78\,P_{HV} - 0.31\,P_g^2}{100}$$

where

P_{HV} = percentage heavy vehicles in the corresponding movement group (%), and

P_g = approach grade for the corresponding movement group (%).

This factor applies to heavy vehicle percentages up to 50% and grades ranging from –4.0% to +10.0%. This factor does not address local buses that stop in the intersection area.

Adjustment for Parking

The parking adjustment factor f_p accounts for the frictional effect of a parking lane on flow in the lane group adjacent to the parking lane. It also accounts for the occasional blocking of an adjacent lane by vehicles moving into and out of parking spaces. If no parking is present, then this factor has a value of 1.00. If parking is present, then the value of this factor is computed with Equation 19-11.

Equation 19-11

$$f_p = \frac{N - 0.1 - \frac{18\,N_m}{3{,}600}}{N} \geq 0.050$$

where

N_m = parking maneuver rate adjacent to lane group (maneuvers/h), and

N = number of lanes in lane group (ln).

The parking maneuver rate corresponds to parking areas directly adjacent to the lane group and within 250 ft upstream of the stop line. A practical upper limit of 180 maneuvers/h should be maintained with Equation 19-11. A minimum value of f_p from this equation is 0.050. Each maneuver (either in or out) is assumed to block traffic in the lane next to the parking maneuver for an average of 18 s.

The factor applies only to the lane group adjacent to the parking. On a one-way street with a single-lane lane group, the number of maneuvers used is the total for both sides of the lane group. On a one-way street with two or more lane groups, the factor is calculated separately for each lane group and is based on the number of maneuvers adjacent to the group. Parking conditions with zero maneuvers have an impact different from that of a no-parking situation.

Adjustment for Bus Blockage

The bus-blockage adjustment factor f_{bb} accounts for the impact of local transit buses that stop to discharge or pick up passengers at a near-side or far-side bus stop within 250 ft of the stop line (upstream or downstream). Values of this factor are computed with Equation 19-12.

$$f_{bb} = \frac{N - \frac{14.4\, N_b}{3{,}600}}{N} \geq 0.050$$

Equation 19-12

where N is the number of lanes in lane group (ln), and N_b is the bus stopping rate on the subject approach (buses/h).

This factor should be used only when stopping buses block traffic flow in the subject lane group. A practical upper limit of 250 buses/h should be maintained with Equation 19-12. A minimum value of f_{bb} from this equation is 0.050. The factor used here assumes an average blockage time of 14.4 s during a green indication.

Adjustment for Area Type

The area type adjustment factor f_a accounts for the inefficiency of intersections in CBDs relative to those in other locations. When used, it has a value of 0.90.

Use of this factor should be determined on a case-by-case basis. This factor is not limited to designated CBD areas, nor does it need to be used for all CBD areas. Instead, it should be used in areas where the geometric design and the traffic or pedestrian flows, or both, are such that vehicle headways are significantly increased.

Adjustment for Lane Utilization

The input lane utilization adjustment factor is used to estimate saturation flow rate for a lane group with more than one exclusive lane. If the lane group has one shared lane or one exclusive lane, then this factor is 1.0.

Adjustment for Right Turns

The right-turn adjustment factor f_{RT} is intended primarily to reflect the effect of right-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 19-13.

Equation 19-13

$$f_{RT} = \frac{1}{E_R}$$

where E_R is the equivalent number of through cars for a protected right-turning vehicle (= 1.18).

If the right-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Section 3 of Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians and bicycles on right-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Left Turns

The left-turn adjustment factor f_{LT} is intended primarily to reflect the effect of left-turn path geometry on saturation flow rate. The value of this adjustment factor is computed with Equation 19-14.

Equation 19-14

$$f_{LT} = \frac{1}{E_L}$$

where E_L is the equivalent number of through cars for a protected left-turning vehicle (= 1.05).

If the left-turn movement shares a lane with another movement or has permitted operation, then the procedure described in Section 3 of Chapter 31 should be used to compute the adjusted saturation flow rate for the shared-lane lane group. The effect of pedestrians on left-turn saturation flow rate is considered in a separate adjustment factor.

Adjustment for Pedestrians and Bicycles

The procedure to determine the left-turn pedestrian–bicycle adjustment factor f_{Lpb} and the right-turn pedestrian–bicycle adjustment factor f_{Rpb} is based on the concept of conflict zone occupancy, which accounts for the conflict between turning vehicles, pedestrians, and bicycles. Relevant conflict zone occupancy takes into account whether the opposing vehicle flow is also in conflict with the left-turn movement. The proportion of green time in which the conflict zone is occupied is determined as a function of the relevant occupancy and the number of receiving lanes for the turning vehicles. A procedure for computing these factors is provided in Section 2 of Chapter 31. These factors have a value of 1.0 if no pedestrians or bicycles are present.

Adjustment for Work Zone Presence

The adjustment factor for work zone presence f_{wz} is used to evaluate the effect of work zone presence on saturation flow rate. This factor addresses the case in which the work zone is located on the intersection approach. The work zone is considered to be on the intersection approach if some (or all) of the work zone is located between the stop line and a point 250 ft upstream of the stop line. A procedure for computing this factor is provided in Section 2 of Chapter 31, Signalized Intersections: Supplemental. The factor has a value of 1.0 if no work zone is present.

Adjustment for Downstream Lane Blockage

The adjustment factor for downstream lane blockage f_{ms} is used to evaluate the effect of a downstream lane closure on saturation flow rate. A downstream lane closure is a closure located downstream of the subject intersection. The factor is applied only to those lane groups entering the segment on which the closure is present. The lane closure can be associated with a work zone or special event. A procedure for computing this factor is provided in Section 3 of Chapter 30, Urban Street Segments: Supplemental. The factor has a value of 1.0 if no downstream lane blockage is present.

Adjustment for Sustained Spillback

The adjustment factor for sustained spillback f_{sp} is used to evaluate the effect of spillback from the downstream intersection. When spillback occurs, its effect is quantified as a reduction in the saturation flow rate of upstream lane groups entering the segment. A procedure is described in Section 3 of Chapter 29, Urban Street Facilities: Supplemental, for evaluating urban street facilities that experience spillback on one or more segments during the analysis period. The calculation of the adjustment factor for spillback is one part of this procedure. The factor has a value of 1.0 if no spillback occurs.

Step 5: Determine Proportion Arriving During Green

Control delay and queue size at a signalized intersection are highly dependent on the proportion of vehicles that arrive during the green and red signal indications. Delay and queue size are smaller when a larger proportion of vehicles arrive during the green indication. Equation 19-15 is used to compute this proportion for each lane group.

$$P = R_p(g/C)$$

Equation 19-15

where all variables are as previously defined.

Equation 19-15 requires knowledge of the effective green time g and cycle length C. These values are known for pretimed operation. If the intersection is not pretimed, then the average phase time and cycle length must be calculated by the procedures described in the next step.

A procedure is described in Section 3 of Chapter 30 that can be used to estimate the arrival flow profile for an intersection approach when this approach is evaluated as part of an urban street segment. The procedure uses the profile to compute the proportion of arrivals during the green indication.

Step 6: Determine Signal Phase Duration

The duration of a signal phase depends on the type of control used at the subject intersection. If the intersection has pretimed control, then the phase duration is an input, and the evaluation continues with Step 7. If the phase duration is unknown, then the pretimed phase duration procedure in Section 2 of Chapter 31 can be used to estimate the pretimed phase duration.

If the intersection has actuated control, then the actuated phase duration procedure in Section 2 of Chapter 31 is used in this step to estimate the average duration of an actuated phase. This procedure distinguishes between actuated, noncoordinated, and coordinated control types.

Step 7: Determine Capacity and Volume-to-Capacity Ratio

The capacity of a given lane group serving one traffic movement, and for which there are no permitted left-turn movements, is defined by Equation 19-16.

Equation 19-16

$$c = N\, s\, \frac{g}{C}$$

where c is the capacity (veh/h), and all other variables are as previously defined. Equation 19-16 cannot be used to calculate the capacity of a shared-lane lane group or a lane group with permitted operation because these lane groups have other factors that affect their capacity. Chapter 31 provides a procedure for estimating the capacity of these types of lane groups.

The volume-to-capacity ratio for a lane group is defined as the ratio of the lane group volume and its capacity. It is computed with Equation 19-17.

Equation 19-17

$$X = \frac{v}{c}$$

where

X = volume-to-capacity ratio,

v = demand flow rate (veh/h), and

c = capacity (veh/h).

The critical intersection volume-to-capacity ratio is also computed during this step. Guidelines for computing this ratio are provided in Section 4, Extensions to the Motorized Vehicle Methodology.

Step 8: Determine Delay

The delay calculated in this step represents the average control delay experienced by all vehicles that arrive during the analysis period. It includes any delay incurred by these vehicles that are still in queue after the analysis period ends. The control delay for a given lane group is computed with Equation 19-18.

Equation 19-18

$$d = d_1 + d_2 + d_3$$

where

d = control delay (s/veh),

d_1 = uniform delay (s/veh),

d_2 = incremental delay (s/veh), and

d_3 = initial queue delay (s/veh).

Chapter 31 describes a technique for measuring control delay in the field.

A. Compute Uniform Delay

The uniform delay for a given lane group serving one traffic movement, and for which there are no permitted movements, is computed by using Equation 19-19 with Equation 19-20 and Equation 19-21.

$$d_1 = PF \frac{0.5\,C(1-g/C)^2}{1-[\min(1,X)\,g/C]}$$

Equation 19-19

with

$$PF = \frac{1-P}{1-g/C} \times \frac{1-y}{1-\min(1,X)\,P} \times \left[1 + y\frac{1-P\,C/g}{1-g/C}\right]$$

Equation 19-20

$$y = \min(1,X)\,g/C$$

Equation 19-21

where

PF = progression adjustment factor,

y = flow ratio,

P = proportion of vehicles arriving during the green indication (decimal),

g = effective green time (s), and

C = cycle length (s).

Equation 19-19 does not provide an accurate estimate of uniform delay for a shared-lane lane group or a lane group with permitted operation because these lane groups have other factors that affect their delay. Also, this equation does not provide an accurate estimate of uniform delay when there is an initial queue present for one or more intersection traffic movements. Section 4, Extensions to the Motorized Vehicle Methodology, describes a procedure for accurately estimating uniform delay when any of these conditions is present.

B. Compute Initial Queue Delay

The initial queue delay term accounts for the additional uniform delay incurred due to an initial queue. This queue is a result of unmet demand in the previous time period. It does *not* include delay to any vehicles that may be in queue due to the random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. The initial queue delay equals 0.0 s/veh when there is no initial queue present at the start of the analysis period for any intersection lane group. A procedure for estimating the associated delay for lane groups with an initial queue present is provided in Section 4, Extensions to the Motorized Vehicle Methodology.

C. Compute Incremental Delay Factor

The equation for computing incremental delay includes a variable that accounts for the effect of controller type on delay. This variable is referred to as the incremental delay factor k. It varies in value from 0.04 to 0.50. A factor value of 0.50 is recommended for pretimed phases, coordinated phases, and phases set to "recall-to-maximum."

An actuated phase has the ability to adapt its green interval duration to serve the demand on a cycle-by-cycle basis and, thereby, to minimize the frequency of cycle failure. Only when the green is extended to its maximum limit is this capability curtailed. This influence of actuated operation on delay is accounted for in Equation 19-22 through Equation 19-25.

Equation 19-22

$$k = (1 - 2\,k_{min})(v/c_a - 0.5) + k_{min} \leq 0.50$$

with

Equation 19-23

$$k_{min} = -0.375 + 0.354\,PT - 0.0910\,PT^2 + 0.00889\,PT^3 \geq 0.04$$

Equation 19-24

$$c_a = \frac{g_a\,s\,N}{C}$$

Equation 19-25

$$g_a = G_{max} + Y + R_c - l_1 - l_2$$

where

k = incremental delay factor,

c_a = available capacity for a lane group served by an actuated phase (veh/h),

k_{min} = minimum incremental delay factor,

PT = passage time setting (s),

G_{max} = maximum green setting (s), and

g_a = available effective green time (s).

As indicated by this series of equations, the factor value depends on the maximum green setting and the passage time setting for the phase that controls the subject lane group. Research indicates shorter passage times result in a lower value of k (and lower delay), provided the passage time is not so short that the phase terminates before the queue is served (*11*).

D. Compute Incremental Delay

Incremental delay consists of two delay components. One component accounts for delay due to the effect of random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity. This delay is evidenced by the occasional overflow queue at the end of the green interval (i.e., cycle failure). The second component accounts for delay due to a sustained oversaturation during the analysis period. This delay occurs when aggregate demand during the analysis period exceeds aggregate capacity. It is sometimes referred to as the deterministic delay component and is shown as variable $d_{2,d}$ in Exhibit 19-21.

Exhibit 19-21 illustrates the queue growth that occurs as vehicles arrive at a demand flow rate v during analysis period T, which has capacity c. The deterministic delay component is represented by the triangular area bounded by the thick line and is associated with an average delay per vehicle represented by the variable $d_{2,d}$. The last vehicle to arrive during the analysis period is shown to clear the queue t_c hours after the start of the analysis period. The average queue size associated with this delay is shown in the exhibit as $Q_{2,d}$. The queue present at the end of the analysis period [= $T\,(v - c)$] is referred to as the residual queue.

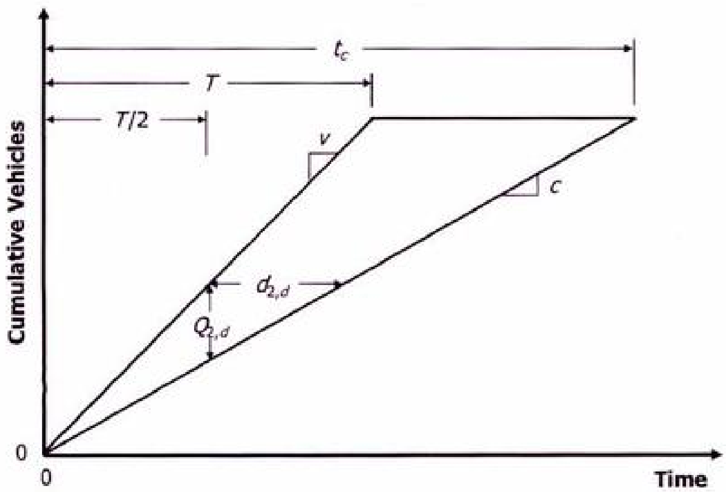


Exhibit 19-21
Cumulative Arrivals and Departures During an Oversaturated Analysis Period

The incremental delay term accounts for delay due to random variation in the number of arrivals on a cycle-by-cycle basis. It also accounts for delay caused by demand exceeding capacity during the analysis period. The amount by which demand exceeds capacity during the analysis period is referred to as unmet demand. The incremental delay equation was derived by using an assumption of no initial queue due to unmet demand in the preceding analysis period. Equation 19-26, with Equation 19-27, is used to compute incremental delay.

$$d_2 = 900\,T\left[(X_A - 1) + \sqrt{(X_A - 1)^2 + \frac{8\,k\,I\,X_A}{c_A\,T}}\right]$$

Equation 19-26

with

$$X_A = v/c_A$$

Equation 19-27

where X_A is the average volume-to-capacity ratio, c_A is the average capacity (veh/h), and all other variables are as previously defined. The variable c_A is not the same as the variable c_a, the latter of which is computed in Part C of Step 8.

If no lane group at the intersection has an initial queue, then the average lane group capacity c_A is equal to the capacity c computed in Step 7 (i.e., $c_A = c$). If one or more lane groups have an initial queue, then the procedure described in Section 4 is used to compute capacity c_A.

The incremental delay term is valid for all values of X_A, including highly oversaturated lane groups.

E. Compute Lane Group Control Delay

The uniform delay, incremental delay, and initial queue delay values computed in the previous steps are added (see Equation 19-18) to estimate the control delay for the subject lane group.

F. Compute Aggregated Delay Estimates

It is often desirable to compute the average control delay for the intersection approach. This aggregated delay represents a weighted average delay, where each lane group delay is weighted by the lane group demand flow rate. The approach control delay is computed with Equation 19-28.

Equation 19-28

$$d_{A,j} = \frac{\sum_{i=1}^{m_j} d_i \, v_i}{\sum_{i=1}^{m_j} v_i}$$

where

$d_{A,j}$ = approach control delay for approach j (s/veh),

d_i = control delay for lane group i (s/veh),

m_j = number of lane groups on approach j, and

all other variables are as previously defined. The summation terms in Equation 19-28 represent the sum for all lane groups on the subject approach.

Similarly, intersection control delay is computed with Equation 19-29.

Equation 19-29

$$d_I = \frac{\sum d_i \, v_i}{\sum v_i}$$

where d_I is the intersection control delay (s/veh). The summation terms in Equation 19-29 represent the sum for all lane groups at the subject intersection.

Unsignalized movements at the signalized intersection should also be considered when an aggregated delay estimate is computed. Inclusion of these movements should be handled as follows:

- Delay of unsignalized movements should be included in the approach and intersection aggregate delay calculations of Equation 19-28 and Equation 19-29, except for special cases that are properly annotated in the results.
- When the delay of unsignalized movements is included in the approach and intersection averages, whether zero or nonzero, the aggregate delay that results must be annotated with a footnote that indicates this unsignalized delay inclusion.
- When the delay of unsignalized movements is not included in the aggregate totals [i.e., it is not included in either the numerator (volume × delay) or the denominator (volume) of Equation 19-28 or Equation 19-29], this exclusion of unsignalized delay must be clearly represented by a footnote that indicates this unsignalized delay exclusion.

Step 9: Determine LOS

Exhibit 19-8 is used to determine the LOS for each lane group, each approach, and the intersection as a whole. LOS is an indication of the acceptability of delay levels to motorists at the intersection. It can also indicate an unacceptable oversaturated operation for individual lane groups.

Step 10: Determine Queue Storage Ratio

A procedure is described in Section 4 of Chapter 31 for estimating the back-of-queue size and the queue storage ratio. The back-of-queue position is the position of the vehicle stopped farthest from the stop line during the cycle as a consequence of the display of a red signal indication. The back-of-queue size depends on the arrival pattern of vehicles and on the number of vehicles that do not clear the intersection during the previous cycle.

The queue storage ratio represents the proportion of the available queue storage distance that is occupied at the point in the cycle when the back-of-queue position is reached. If this ratio exceeds 1.0, then the storage space will overflow, and queued vehicles may block other vehicles from moving forward.

Interpretation of Results

The computations discussed in the previous steps result in the estimation of control delay and LOS for each lane group, for each approach, and for the intersection as a whole. They also produce a volume-to-capacity ratio for each lane group and a critical intersection volume-to-capacity ratio. This subsection provides some useful interpretations of these performance measures.

Level of Service

In general, LOS is an indication of the *general* acceptability of delay to drivers. In this regard, it should be remembered that what might be acceptable in a large city is not necessarily acceptable in a smaller city or rural area.

Intersection LOS must be interpreted with caution. It can suggest acceptable operation of the intersection when in reality certain lane groups (particularly those with lower volumes) are operating at an unacceptable LOS but are masked at the intersection level by the acceptable performance of higher-volume lane groups. The analyst should always verify that each lane group is providing acceptable operation and consider reporting the LOS for the poorest-performing lane group as a means of providing context to the interpretation of intersection LOS.

Volume-to-Capacity Ratio

In general, a volume-to-capacity ratio greater than 1.0 is an indication of actual or potential breakdown. In such cases, a multiple-period analysis is advised for this condition. This analysis would encompass all consecutive periods in which a residual queue is present.

The critical intersection volume-to-capacity ratio is useful in evaluating the intersection from a capacity-only perspective. It is possible to have a critical intersection volume-to-capacity ratio of less than 1.0 and still have individual movements oversaturated within the signal cycle. If this situation occurs, then the cycle time is generally not appropriately allocated among the phases. Reallocation of the cycle time should be considered, so that additional time is given to the phases serving those lane groups with a volume-to-capacity ratio greater than 1.0.

A critical intersection volume-to-capacity ratio greater than 1.0 indicates the overall signal timing and geometric design provide inadequate capacity for the given demand flows. Improvements that might be considered include the following:

- Basic changes in intersection geometry (i.e., change in the number or use of lanes),

- Increase in signal cycle length if it is determined to be too short, and
- Changes in signal phase sequence or timing.

Local guidelines should always be consulted before potential improvements are developed.

Fully actuated control is intended to allocate cycle time dynamically to movements on the basis of demand and, thereby, maintain efficient operation on a cycle-by-cycle basis. The critical intersection volume-to-capacity ratio can provide an indication of this efficiency. In general, this ratio will vary between 0.85 and 0.95 for most actuated intersections, with lower values in this range more common for intersections having multiple detectors in the through traffic lanes. A ratio less than 0.85 may indicate excessive green extension by random arrivals, and the analyst may consider reducing passage time, minimum green, or both. A ratio more than 0.95 may indicate frequent phase termination by max-out and limited ability of the controller to reallocate cycle time dynamically on the basis of detected demand. Increasing the maximum green may improve operation in some instances; however, it may also degrade operation when phase flow rates vary widely (because green extension is based on total flow rate served by the phase, not flow rate per lane).

For semiactuated and coordinated-actuated control, the critical intersection volume-to-capacity ratio can vary widely because of the nonactuated nature of some phases. The duration of these phases may not be directly related to their associated demand; instead, it may be dictated by coordination timing or the demand for the other phases. A critical intersection volume-to-capacity ratio that exceeds 0.95 has the same interpretation as offered previously for fully actuated control.

The critical intersection volume-to-capacity ratio can be misleading when it is used to evaluate the overall sufficiency of the intersection geometry, as is often required in planning applications. The problem is that low flow rates dictate the need for short cycle lengths to minimize delay. However, the equation for calculating this ratio indicates the desired shorter cycle length produces a higher volume-to-capacity ratio. Therefore, a relatively large value for this ratio (provided it is less than 1.0) is not a certain indication of poor operation. Rather, it means closer attention must be paid to the adequacy of phase duration and queue size, especially for the critical phases.

Volume-to-Capacity Ratio and Delay Combinations

In some cases, delay is high even when the volume-to-capacity ratio is low. In these situations, poor progression, a notably long cycle length, or an inefficient phase plan is generally the cause. When the intersection is part of a coordinated system, the cycle length is determined by system considerations, and alterations at individual intersections may not be practical.

It is possible for delay to be at acceptable levels even when the volume-to-capacity ratio is high. This situation can occur when some combination of the following conditions exists: the cycle length is relatively short, the analysis period is short, the lane group capacity is high, and there is no initial queue. If a residual queue is created in this scenario, then a multiple-period analysis is necessary to gain a true picture of the delay.

When both delay levels and volume-to-capacity ratios are unacceptably high, the situation is critical. In such situations, the delay may increase rapidly with small changes in demand. The full range of potential geometric and signal design changes should be considered in the search for improvements.

In summary, unacceptable delay can exist when capacity is a problem as well as when capacity is adequate. Further, acceptable delay levels do not automatically ensure capacity is sufficient. Delay and capacity are complex variables that are influenced by a wide range of traffic, roadway, and signalization conditions. The methodology presented here can be used to estimate these performance measures, identify possible problems, and assist in developing alternative improvements.

4. EXTENSIONS TO THE MOTORIZED VEHICLE METHODOLOGY

CRITICAL INTERSECTION VOLUME-TO-CAPACITY RATIO

Overview

A useful concept for analyzing signalized intersections is the critical intersection volume-to-capacity ratio X_c. This ratio is computed by using Equation 19-30 with Equation 19-31.

Equation 19-30

$$X_c = \left(\frac{C}{C-L}\right)\sum_{i \in ci} y_{c,i}$$

with

Equation 19-31

$$L = \sum_{i \in ci} l_{t,i}$$

where

X_c = critical intersection volume-to-capacity ratio,

C = cycle length (s),

$y_{c,i}$ = critical flow ratio for phase $i = v_i/(N\, s_i)$,

$l_{t,i}$ = phase i lost time = $l_{1,i} + l_{2,i}$ (s),

ci = set of critical phases on the critical path, and

L = cycle lost time (s).

The summation term in each of these equations represents the sum of a specific variable for the set of critical phases. A critical phase is one phase of a set of phases that occurs in sequence and whose combined flow ratio is the largest for the signal cycle. The critical path and critical phases are identified by mapping traffic movements to a dual-ring phase diagram, as shown in Exhibit 19-2.

Equation 19-30 is based on the combined assumption that each critical phase has the same volume-to-capacity ratio and that this ratio is equal to the critical intersection volume-to-capacity ratio. This assumption is valid when the effective green duration for each critical phase i is proportional to $y_{c,i}/\Sigma(y_{c,i})$. When this assumption holds, the volume-to-capacity ratio for each noncritical phase is less than or equal to the critical intersection volume-to-capacity ratio.

Identifying Critical Lane Groups and Critical Flow Ratios

Calculation of the critical intersection volume-to-capacity ratio requires identification of the critical phases. This identification begins by mapping all traffic movements to a dual-ring diagram.

Next, the lane group flow ratio is computed for each lane group served by the phase. If a lane group is served only during one phase, then its flow ratio is computed as the lane group flow rate (per lane) divided by the lane group saturation flow rate [i.e., $v_i/(N\, s_i)$]. If a lane group is served during multiple phases (e.g., protected-permitted), then a flow ratio is computed for each phase. Specifically, the demand flow rate and saturation flow rate that occur during a given phase are used to compute the lane group flow ratio for that phase.

If the lane group is served in a permitted manner, then the saturation flow rate s_i used to determine the flow ratio is an average for the permitted green period. For left turns, it is computed with Equation 31-59 in Chapter 31 (with E_{L2} and E_{L1} substituted for $E_{L2,m}$ and $E_{L1,m}$, respectively) and the instructions that follow this equation as they relate to shared or exclusive lane assignment. This equation applies to lane groups served as permitted-only and to lane groups served during the permitted phase of protected-permitted operation. For right turns, Equation 31-61 is used (with E_R substituted for $E_{R,m}$).

If the lane group is served by protected-permitted operation, then its volume v_i must be apportioned to the protected and permitted phases. To accomplish this apportionment, it is appropriate to consider the phase that is displayed first to be fully saturated by turning traffic and to apply any residual flow to the phase that is displayed second. In this manner, the volume assigned to the first phase is the smaller of the phase capacity or the demand volume, and any unassigned volume goes to the second phase.

Next, the phase flow ratio is determined from the flow ratio of each lane group served during the phase. The phase flow ratio represents the largest flow ratio of all lane groups served.

Next, the diagram is evaluated to identify the critical phases. The phases that occur between one barrier pair are collectively evaluated to determine the critical phases. This evaluation begins with the pair in Ring 1 and proceeds to the pair in Ring 2. Each ring represents one possible critical path. The phase flow ratios are added for each phase pair in each ring. The larger of the two ring totals represents the critical path, and the corresponding phases represent the critical phases for the barrier pair.

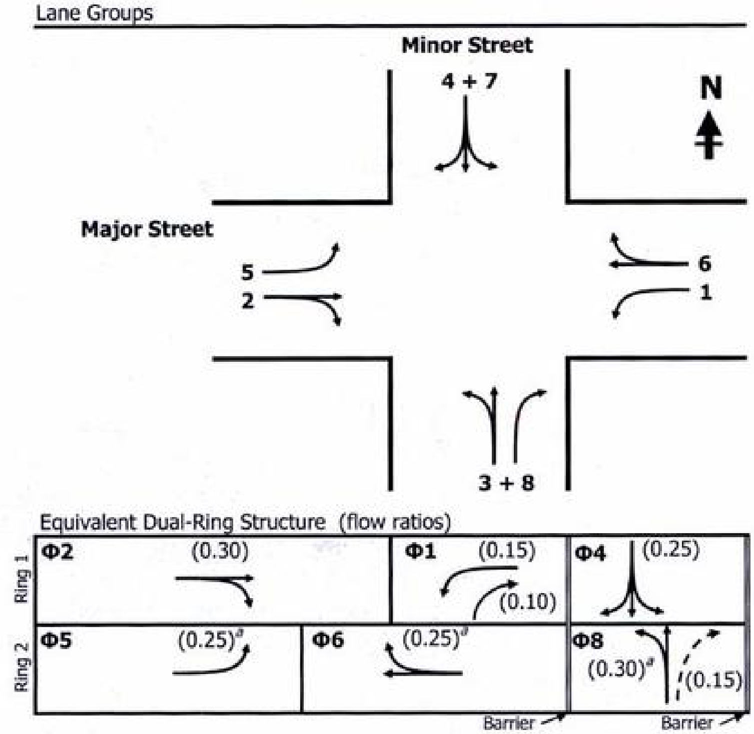
Finally, the process is repeated for the phases between the other barrier pair. One critical flow rate is defined for each barrier pair by this process. These two values are then added to obtain the sum of the critical flow ratios used in Equation 19-30. The lost time associated with each of the critical phases is added to yield the cycle lost time L.

The procedure for the basic intersection case is explained in the next few paragraphs by using an example intersection. A variation of this procedure that applies when protected-permitted operation is used is described after the basic case is described.

Basic Case

For the basic case, consider an intersection with a lead–lag phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 19-22. The northbound right turn is provided an exclusive lane and a green arrow indication that displays concurrently with the complementary left-turn phase on the major street. Each of the left-turn movements on the major street is served with a protected phase.

Exhibit 19-22
Critical Path Determination with Protected Left-Turn Phases



Note: [a] Critical flow ratio.

Phases 4 and 8 represent the only phases between the barrier pair serving the minor-street movements. Inspection of the flow ratios provided in the exhibit indicates Phase 8 has two lane-group flow rates. The larger flow rate corresponds to the shared left-turn and through movement. Thus, the phase flow ratio for Phase 8 is 0.30. The phase flow ratio for Phase 4 is 0.25. Of the two phases, the larger phase flow ratio is associated with Phase 8 (= 0.30), so it represents the critical phase for this barrier pair.

Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. The phase flow ratio of Phase 1 is 0.15, on the basis of the left-turn lane group flow rate.

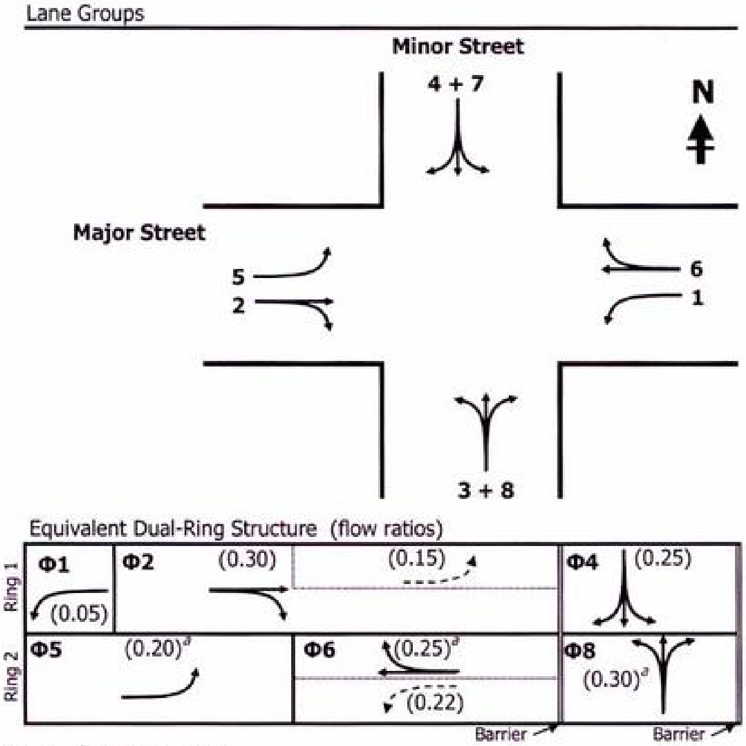
There are two possible critical paths through the major-street phase sequence. One path is associated with Phases 1 and 2 (i.e., Ring 1), and the other path is associated with Phases 5 and 6 (i.e., Ring 2). The total phase flow ratio for the Ring 1 path is 0.30 + 0.15 = 0.45. The total phase flow ratio for the Ring 2 path is 0.25 + 0.25 = 0.50. The latter total is larger and, hence, represents the critical path. It identifies Phases 5 and 6 as the critical phases. Thus, the sum of critical flow ratios for the cycle is 0.80 (= 0.30 + 0.50).

One increment of phase lost time l_t is associated with each phase on the critical path. Thus, the cycle lost time L is computed as the sum of the lost time for each of Phases 5, 6, and 8.

Special Case: Protected-Permitted Left-Turn Operation

For the special case, consider an intersection with a lead–lead phase sequence on the major street and a permitted-only sequence on the minor street, as shown in Exhibit 19-23. The left-turn movements on the major street operate in the protected-permitted mode. Phases 4 and 8 represent the only phases between one barrier pair. They serve the minor-street lane groups. By inspection of the flow ratios provided in the exhibit, Phase 8 has the highest flow ratio (= 0.30) of the two phases and represents the critical phase for this barrier pair.

Exhibit 19-23
Critical Path Determination with Protected-Permitted Left-Turn Operation



Note: [a] Critical flow ratio.

Phases 1, 2, 5, and 6 represent the phases between the other barrier pair. They serve the major-street approaches. Each left-turn lane group is shown to be served during two phases—once during the left-turn phase and once during the phase serving the adjacent through movement. The flow ratio for each of the four left-turn service periods is shown in Exhibit 19-23. The following rules define the possible critical paths through this phase sequence:

1. One path is associated with Phases 1 and 2 in Ring 1 (0.35 = 0.05 + 0.30).
2. One path is associated with Phases 5 and 6 in Ring 2 (0.45 = 0.20 + 0.25).
3. If a lead–lead or lag–lag phase sequence is used, then one path is associated with (*a*) the left-turn phase with the larger flow ratio and (*b*) the through phase that permissively serves the same left-turn lane group. Sum the protected and permitted left-turn flow ratios on this path (0.35 = 0.20 + 0.15).
4. If a lead–lag phase sequence is used, then one path is associated with (*a*) the leading left-turn phase, (*b*) the lagging left-turn phase, and (*c*) the

controlling through phase (see discussion to follow). Sum the two protected left-turn flow ratios and the one controlling permitted left-turn flow ratio on this path.

If a lead–lag phase sequence is used, each of the through phases that permissively serve a left-turn lane group is considered in determining the controlling through phase. If both through phases have a permitted period, then there are two through phases to consider. The controlling through phase is that phase with the larger permitted left-turn flow ratio. For example, if Phase 1 were shown to lag Phase 2 in Exhibit 19-23, then Phase 6 would be the controlling through phase because the permitted left-turn flow ratio of 0.22 exceeds 0.15. The critical path for this phase sequence would be 0.47 (= 0.20 + 0.22 + 0.05).

The first three rules in the preceding list apply to the example intersection. The calculations are shown for each path in parentheses in the previous list of rules. The total flow ratio for the path in Ring 2 is largest (= 0.45) and, hence, represents the critical path. It identifies Phases 5 and 6 as the critical phases. Thus, the sum of critical flow ratios for the cycle is 0.75 (= 0.30 + 0.45).

If Rule 3 in the preceding list applies, then the only lost time incurred is the start-up lost time l_1 associated with the first critical phase and the clearance lost time l_2 associated with the second critical phase. If Rule 1, 2, or 4 applies, then one increment of phase lost time l_t is associated with each critical phase. Rule 2 applies for the example, so the cycle lost time L is computed as the sum of the lost time for each of Phases 5, 6, and 8.

Two flow ratios are associated with Phase 6 in this example. Both flow ratios are shown possibly to dictate the duration of Phase 6 (this condition does not hold for Phase 2 because of the timing of the left-turn phases). This condition is similar to that for the northbound right-turn movement in Phase 1 of Exhibit 19-22, and the treatment is the same. That is, both flow ratios are considered in defining the phase flow ratio for Phase 6.

UNIFORM DELAY CALCULATION USING QUEUE ACCUMULATION POLYGON

Overview

This subsection describes a procedure for calculating uniform delay. This incremental queue accumulation procedure (*21*, *22*) is sufficiently general that it can be applied to any lane group, regardless of whether the lane group is shared or exclusive or served with a protected, permitted, or protected-permitted operation.

The incremental queue accumulation procedure models arrivals and departures as they occur during the average cycle. Specifically, it considers arrival rates and departure rates as they may occur during one or more effective green periods. The rates and resulting queue size can be shown in a queue accumulation polygon, such as that shown previously in Exhibit 19-7. The procedure decomposes the resulting polygon into an equivalent set of trapezoids or triangles for the purpose of delay estimation.

Polygon Construction

The key criterion for constructing a trapezoid or triangle is that the arrival and departure rates must be effectively constant during the associated time period. This process is illustrated in Exhibit 19-24 for a lane group having two different departure rates during the effective green period.

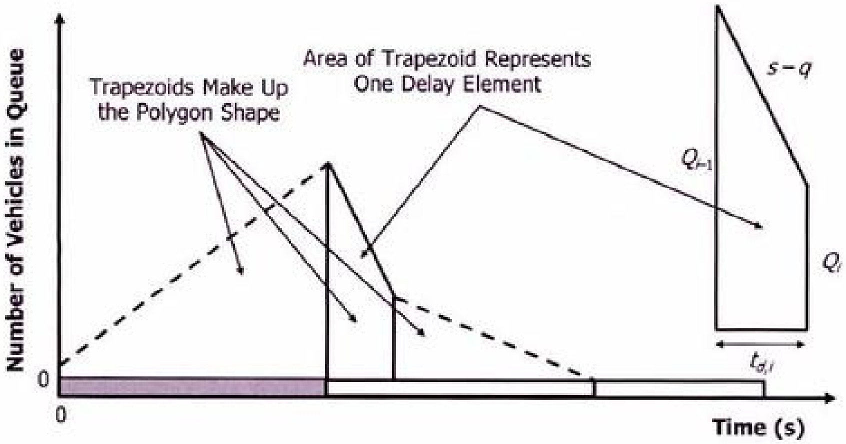


Exhibit 19-24
Decomposition of Queue Accumulation Polygon

Construction of the queue accumulation polygon requires that the arrival flow rate not exceed the phase capacity. If the arrival flow rate exceeds capacity, then it is set to equal the capacity for the purpose of constructing the polygon. The queue can be assumed to equal zero at the end of the protected phase, and the polygon construction process begins at this point in the cycle. Once the polygon is constructed, this assumption must be checked. If the ending queue is not zero, then a second polygon is constructed with this ending queue as the starting queue for the first interval.

Construction of the queue accumulation polygon requires converting all flow rate variables to common units of vehicles per second per lane. This conversion is implicit for all flow rate variables shown in the exhibits that depict a queue accumulation polygon.

Polygon construction requires identifying points in the cycle at which one of the following two conditions applies:

- The departure rate changes (e.g., due to the start or end of effective green, a change in the saturation flow rate, depletion of the subject queue, depletion of the opposing queue, departure of sneakers), or
- The arrival rate changes (e.g., when a platoon arrival condition changes).

During the intervals of time between these points, the saturation flow rate and arrival flow rate are constant.

The determination of flow-rate change points may require an iterative calculation process when the approach has shared lanes. For example, an analysis of the opposing through movement must be completed to determine the time this movement's queue clears and the subject left-turn lane group can begin its service period. This service period may, in turn, dictate when the permitted left-turn movements on the opposing approach may depart.

The procedure is based on defining arrival rate as having one of two flow states: an arrival rate during the green indication and an arrival rate during the red indication. Further information about when each of these rates applies is described in the discussion for platoon ratio in the Required Data and Sources subsection. The proportion of vehicles arriving during the green indication P is used to compute the arrival flow rate during each flow state. Equation 19-32 and Equation 19-33 can be used to compute these rates.

Equation 19-32

$$q_g = \frac{q\,P}{g/C}$$

and

Equation 19-33

$$q_r = \frac{q\,(1-P)}{1-g/C}$$

where

q_g = arrival flow rate during the effective green time (veh/s),

q_r = arrival flow rate during the effective red time (veh/s),

q = arrival flow rate = $v/3{,}600$ (veh/s),

P = proportion of vehicles arriving during the green indication (decimal), and

g = effective green time (s).

A more detailed description of the procedure for constructing a queue accumulation polygon for lane groups with various lane allocations and operating modes is provided in Section 3 of Chapter 31.

Delay Calculation

The uniform delay is determined by summing the area of the trapezoids or triangles that compose the polygon. The area of a given trapezoid or triangle is determined by first knowing the queue at the start of the interval and then adding the number of arrivals and subtracting the number of departures during the specified time interval. The result of this calculation yields the number of vehicles in queue at the end of the interval. Equation 19-34 illustrates this calculation for interval i.

Equation 19-34

$$Q_i = Q_{i-1} - \left(\frac{s}{3{,}600} - \frac{q}{N}\right) t_{d,i} \geq 0.0$$

where

Q_i = queue size at the end of interval i (veh),

N = number of lanes in the lane group (ln),

s = adjusted saturation flow rate (veh/h/ln), and

$t_{d,i}$ = duration of time interval i during which the arrival flow rate and saturation flow rate are constant (s).

The uniform delay is calculated by using Equation 19-35 with Equation 19-36.

$$d_1 = \frac{0.5 \sum(Q_{i-1} + Q_i)\, t_{t,i}}{q\, C}$$

Equation 19-35

with

$$t_{t,i} = \min\left(t_{d,i}, Q_{i-1}/w_q\right)$$

Equation 19-36

where

d_1 = uniform delay (s/veh),

$t_{t,i}$ = duration of trapezoid or triangle in interval i (s),

w_q = queue change rate (i.e., slope of the upper boundary of the trapezoid or triangle) (veh/s), and

all other variables are as previously defined.

The summation term in Equation 19-35 includes all intervals for which there is a nonzero queue. In general, $t_{t,i}$ will equal the duration of the corresponding interval. However, during some intervals the queue will dissipate, and $t_{t,i}$ will only be as long as the time required for the queue to dissipate ($= Q_{i-1}/w_q$).

INITIAL QUEUE DELAY CALCULATION

Overview

Initial queue delay accounts for the additional delay incurred due to an initial queue. This queue is a result of unmet demand in the previous time period. It does *not* include any vehicles that may be in queue due to random, cycle-by-cycle fluctuations in demand that occasionally exceed capacity.

Exhibit 19-25 illustrates the delay due to an initial queue as a trapezoid shape bounded by thick lines. The average delay per vehicle is represented by the variable d_3. The initial queue size is shown as Q_b vehicles. The duration of time during the analysis period for which the effect of the initial queue is still present is represented by the variable t. This duration is shown to equal the analysis period T in Exhibit 19-25. However, it can be shorter than the analysis period duration for some lower-volume conditions.

Exhibit 19-25 illustrates the case in which the demand flow rate v exceeds the capacity c during the analysis period. In contrast, Exhibit 19-26 and Exhibit 19-27 illustrate alternative cases in which the demand flow rate is less than the capacity.

The remainder of this subsection describes the procedure for computing the initial queue delay for a lane group during a given analysis period.

Exhibit 19-25
Initial Queue Delay with Increasing Queue Size

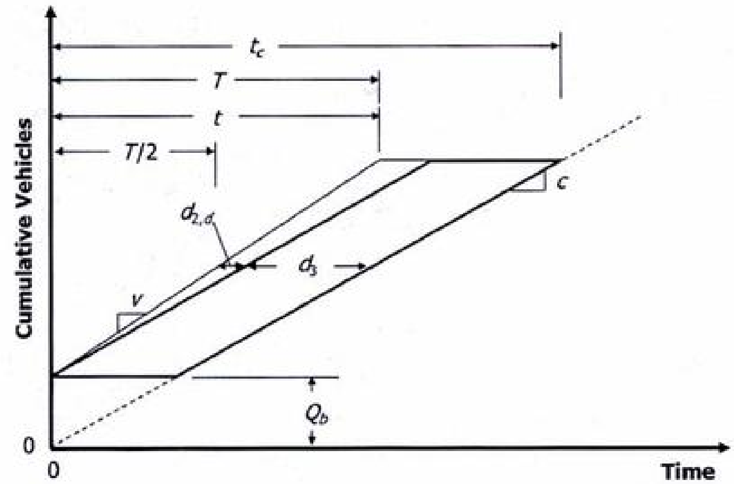


Exhibit 19-26
Initial Queue Delay with Decreasing Queue Size

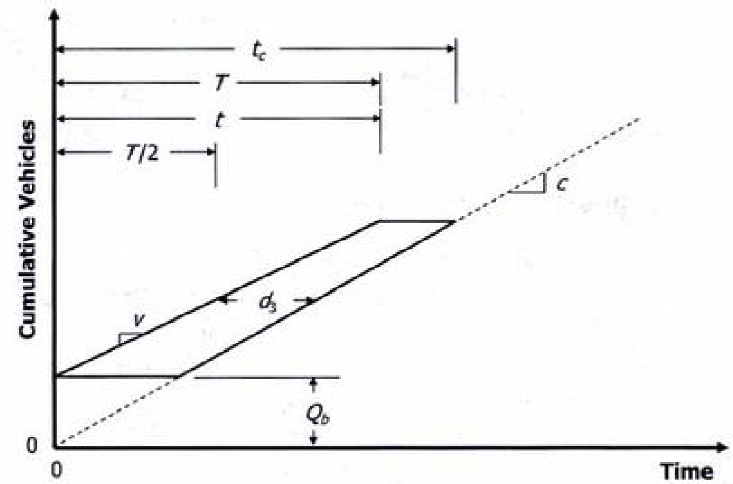
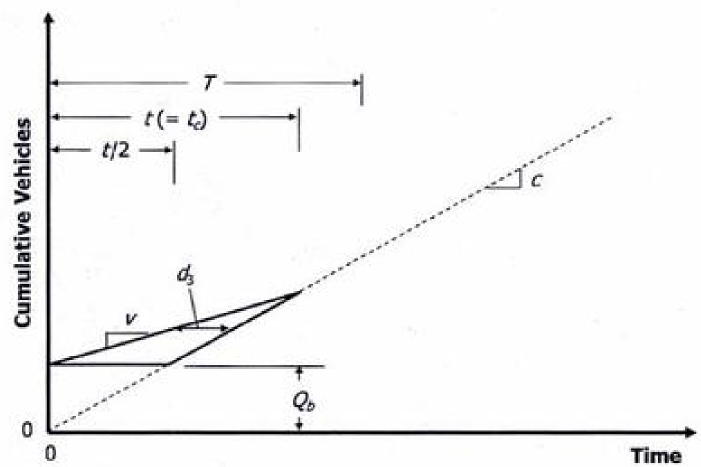


Exhibit 19-27
Initial Queue Delay with Queue Clearing



Computational Steps

A. Initial Queue Analysis

At the start of this step, the initial queue that was input for each movement group needs to be converted to an initial queue for each lane group. When there is a one-to-one correlation between the movement group and the lane group, then the initial queue for the lane group equals the input initial queue for the movement group. When there is a shared lane on a multiple-lane approach, then the input initial queue needs to be distributed among the lane groups that serve the movements sharing the lane. Specifically, the initial queue for each lane group is estimated as being equal to the input initial queue multiplied by the number of lanes in the lane group and divided by the total number of shared and through lanes.

The saturation flow rate, phase duration, capacity, and uniform delay will need to be recomputed for each lane group during this step. When these variables are computed for a lane group with an initial queue, the arrival flow rate for the lane group is inflated such that it equals the lane group capacity (i.e., the actual input demand flow rate is not used). The remaining lane groups will have their arrival flow rate set to equal the smaller of the input demand flow rate or the capacity.

The need to recompute these variables stems from the influence one lane group often has on the operation of other lane groups. This influence is notably adverse when one or more lane groups are operating in a saturated state for a portion of the analysis period. If the saturated lane group represents a conflicting movement to a lane group that includes a permitted left-turn operation, then the left-turn lane group's operation will also be adversely affected for the same time period. Moreover, if the phase serving the lane group is actuated, then its capacity during the saturated state will be different from that of the subsequent unsaturated state.

The uniform delay computed during this step is referred to as the saturated uniform delay. It is computed for each lane group by using the arrival flow rate, capacity, and phase duration determined with the previous guidance.

The duration of unmet demand is calculated in this step for each lane group. Either Equation 19-37 or Equation 19-38 is used for this purpose.

If $v \geq c_s$, then

$$t = T$$

Equation 19-37

If $v < c_s$, then

$$t = Q_b/(c_s - v) \leq T$$

Equation 19-38

where

t = duration of unmet demand in the analysis period (h),

T = analysis period duration (h),

Q_b = initial queue at the start of the analysis period (veh),

v = demand flow rate (veh/h), and

c_s = saturated capacity (veh/h).

For this calculation, the saturated capacity c_s is equal to that obtained from the polygon constructed in this step and is reflective of the phase duration that is associated with saturated operation (due to the initial queue).

Next, the average duration of unmet demand is calculated with Equation 19-39.

Equation 19-39

$$t_a = \frac{1}{N_g} \sum_{i \in N_g} t_i$$

where

t_a = average duration of unmet demand in the analysis period (h),

t_i = duration of unmet demand for lane group i in the analysis period (h), and

N_g = number of lane groups for which t exceeds 0.0 h.

The summation term in Equation 19-39 represents the sum of the t values for only those lane groups that have a value of t that exceeds 0.0 h. The average duration t_a is considered as a single representative value of t for all lane groups that do not have an initial queue.

B. Compute Uniform Delay

The uniform delay computed in Step 8 of the core motorized vehicle methodology is adjusted in this step such that the adjusted uniform delay reflects the presence of the initial queue. Initially, the uniform delay d_1 computed previously is renamed as the baseline uniform delay d_{1b} (i.e., $d_{1b} = d_1$). Next, Equation 19-40 or Equation 19-41 is used to compute the uniform delay for each lane group.

If lane group i has an initial queue, then

Equation 19-40

$$d_{1,i} = d_{s,i}\frac{t_i}{T} + d_{1b,i}\frac{(T - t_i)}{T}$$

If lane group i does not have an initial queue, then

Equation 19-41

$$d_{1,i} = d_{s,i}\frac{t_a}{T} + d_{1b,i}\frac{(T - t_a)}{T}$$

where d_s is the saturated uniform delay (s/veh), d_{1b} is the baseline uniform delay (s/veh), t_i is the duration of unmet demand for lane group i in the analysis period (h), and other variables are as previously defined.

C. Compute Average Capacity

Equation 19-42 and Equation 19-43 are used to compute the average capacity for each lane group.

If lane group i has an initial queue, then

Equation 19-42

$$c_{A,i} = c_{s,i}\frac{t_i}{T} + c_i\frac{(T - t_i)}{T}$$

If lane group i does not have an initial queue, then

$$c_{A,i} = c_{s,i}\frac{t_a}{T} + c_i\frac{(T - t_a)}{T}$$ **Equation 19-43**

where c_A is the average capacity (veh/h).

D. Compute Initial Queue Delay

Equation 19-44 through Equation 19-49 are used to compute the initial queue delay for each lane group.

$$d_3 = \frac{3{,}600}{v\,T}\left(t_A\frac{Q_b + Q_e - Q_{eo}}{2} + \frac{Q_e^2 - Q_{eo}^2}{2\,c_A} - \frac{Q_b^2}{2\,c_A}\right)$$ **Equation 19-44**

with

$$Q_e = Q_b + t_A(v - c_A)$$ **Equation 19-45**

If $v \geq c_A$, then

$$Q_{eo} = T\,(v - c_A)$$ **Equation 19-46**

$$t_A = T$$ **Equation 19-47**

If $v < c_A$, then

$$Q_{eo} = 0.0 \text{ veh}$$ **Equation 19-48**

$$t_A = Q_b/(c_A - v) \leq T$$ **Equation 19-49**

where

t_A = adjusted duration of unmet demand in the analysis period (h),

Q_e = queue at the end of the analysis period (veh),

Q_{eo} = queue at the end of the analysis period when $v \geq c_A$ and $Q_b = 0.0$ (veh), and

all other variables are as previously defined. The queue at the end of the analysis period Q_e is also referred to as the residual queue.

The last vehicle that arrives to an overflow queue during the analysis period will clear the intersection at the time obtained with Equation 19-50.

$$t_c = t_A + Q_e/c_A$$ **Equation 19-50**

where t_c is the queue-clearing time (h).

The queue-clearing time is measured from the start of the analysis period to the time the last arriving vehicle clears the intersection.

5. PEDESTRIAN METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to pedestrians traveling through a signalized intersection.

SCOPE OF THE METHODOLOGY

The overall scope of the three methodologies was provided in Section 2. This section identifies the additional conditions for which the pedestrian methodology is applicable.

- *Target travel modes*. The pedestrian methodology addresses travel by pedestrians walking across one or more legs of a signalized intersection. It is not designed to evaluate the performance of other travel means (e.g., Segway, roller skates).
- *"Typical pedestrian" focus for pedestrian methodology*. The pedestrian methodology is not designed to reflect the perceptions of any particular pedestrian subgroup, such as pedestrians with disabilities. The performance measures obtained from the methodology are not intended to be indicators of a sidewalk's compliance with U.S. Access Board guidelines related to Americans with Disabilities Act requirements. For this reason, they should not be considered as a substitute for an Americans with Disabilities Act compliance assessment of a pedestrian facility.

Spatial Limits

Intersection performance is separately evaluated for each crosswalk and intersection corner with this methodology. *Unless otherwise stated, all variables identified in this subsection are specific to one crosswalk and one corner.* A crosswalk is assumed to exist across each intersection leg unless crossing is specifically prohibited by local ordinance (and signed to this effect).

Performance Measures

Performance measures applicable to the pedestrian travel mode include corner circulation area, crosswalk circulation area, pedestrian delay, and pedestrian LOS score. Pedestrian delay represents the average time a pedestrian waits for a legal opportunity to cross an intersection leg. The LOS score is an indication of the typical pedestrian's perception of the overall crossing experience.

LOS is also considered a performance measure. It is useful for describing intersection performance to elected officials, policy makers, administrators, or the public. LOS is based on the pedestrian LOS score.

The two circulation-area performance measures are based on the concept of pedestrian space. One measure is used to evaluate the circulation area provided to pedestrians while they wait at the corner. The other measure is used to evaluate the area provided while the pedestrian is crossing in the crosswalk.

Circulation area describes the space available to the average pedestrian. A larger area is more desirable from the pedestrian perspective. Exhibit 19-28 can be used to evaluate intersection circulation area performance from the pedestrian perspective.

Exhibit 19-28
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

Limitations of the Methodology

This subsection identifies the known limitations of the pedestrian methodology. If one or more of these limitations is believed to have an important influence on the performance of a specific intersection, then the analyst should consider using alternative methods or tools for the evaluation.

The pedestrian methodology does not account for the effect of the following conditions on the quality of service provided to pedestrians:

- Grades in excess of 2%,
- Presence of railroad crossings,
- Unpaved sidewalk, and
- Free (i.e., uncontrolled) channelized right turn with multiple lanes or high-speed operation.

Pedestrian Flow Conditions

Exhibit 19-29 and Exhibit 19-30 show the variables considered when one corner and its two crosswalks are evaluated. Two flow conditions are illustrated. Condition 1 corresponds to the minor-street crossing that occurs during the major-street through phase. The pedestrians who desire to cross the major street must wait at the corner. Condition 2 corresponds to the major-street crossing that occurs during the minor-street through phase. For this condition, the pedestrians who desire to cross the minor street wait at the corner.

Exhibit 19-29
Condition 1: Minor-Street Crossing

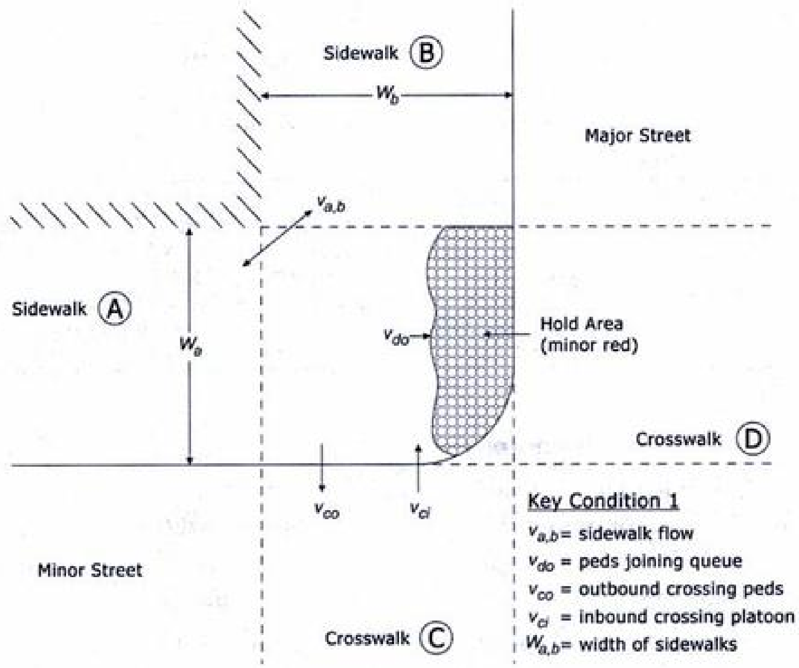
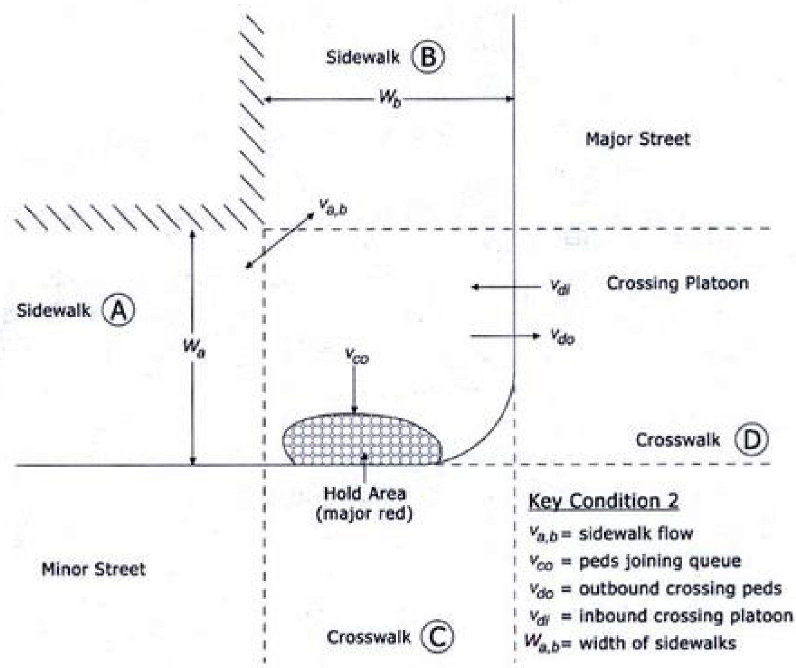


Exhibit 19-30
Condition 2: Major-Street Crossing



REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the pedestrian methodology. These data are listed in Exhibit 19-31. The second column (labeled Basis) of the exhibit indicates whether the input data are needed for each traffic movement, each signal phase, each intersection approach, or the intersection as a whole. The exhibit also lists default values that can be used if local data are not available (*25, 30*).

Exhibit 19-31
Required Input Data, Potential Data Sources, and Default Values for Pedestrian Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
		Traffic Characteristics	
Motorized vehicle demand flow rate (veh/h)	M	Field data, past counts	Must be provided
Right-turn-on-red flow rate (veh/h)	A	Field data, past counts	Must be provided
Permitted left-turn flow rate (veh/h)	M	Field data, past counts	See discussion
Midsegment 85th percentile speed (mi/h)	A	Field data	Speed limit
Pedestrian flow rate (veh/h)	M	Field data, past counts	Must be provided
		Geometric Design	
Number of lanes (ln)	L	Field data, aerial photo	Must be provided
Number of right-turn islands (0, 1, 2)	L	Field data, aerial photo	0
Total walkway width (ft)	A	Field data, aerial photo	Business or office land use: 9.0 ft Residential or industrial land use: 11.0 ft
Crosswalk width (ft)	L	Field data, aerial photo	12 ft
Crosswalk length (ft)	L	Field data, aerial photo	Must be provided
Corner radius (ft)	A	Field data, aerial photo	Trucks and buses in turn volume: 45 ft No trucks or buses in turn volume: 25 ft
		Signal Control	
Walk (s)	P	Field data	Actuated: 7 s Pretimed: green interval minus pedestrian clear
Pedestrian clear (s)	P	Field data	Based on 3.5-ft/s walking speed
Rest in walk (yes or no)	P	Field data	Not enabled
Cycle length (s)	I	Field data	Same as motorized vehicle mode
Yellow change + red clearance (s)[a]	P	Field data	4 s
Duration of phase serving pedestrians (s)	P	Field data	Same as motorized vehicle mode
Pedestrian signal head presence (yes or no)	P	Field data	Must be provided
		Other Data	
Analysis period duration (h)[b]	I	Set by analyst	0.25 h

Notes: M= movement: one value for each left-turn, through, and right-turn movement.
A = approach: one value for the intersection approach.
L = leg: one value for the intersection leg (approach plus departure sides).
P = phase: one value or condition for each signal phase.
I = intersection: one value for the intersection.
[a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.
[b] Analysis period duration is as defined for Exhibit 19-11.

The data elements listed in Exhibit 19-31 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during presentation of the methodology.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 19-31. These data describe the traffic streams traveling through the intersection during the study period. The demand flow rate of motorized vehicles and RTOR flow rate are defined in Section 3 for the motorized vehicle mode.

Permitted Left-Turn Flow Rate

The permitted left-turn flow rate is defined as the count of vehicles that turn left permissively, divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. A permitted left-turn movement can occur with either the permitted or the protected-permitted left-turn mode. For left-turn movements served by the permitted mode, the permitted left-turn flow rate is equal to the left-turn demand flow rate.

Default Value. For left-turn movements served by the protected-permitted mode, the permitted left-turn flow rate should be measured in the field because its value is influenced by many factors. However, a default flow rate can be used if the analysis involves future conditions or if the permitted left-turn flow rate is not known from field data.

The default permitted left-turn flow rate for movements served by the permitted mode is equal to the left-turn demand flow rate.

The default permitted left-turn flow rate for movements served by the protected-permitted mode is equal to the left-turn arrival rate during the permitted period. This arrival rate is estimated as the left-turn flow rate during the effective red time [i.e., $q_r = (1 - P)\, q\, C/r$].

Midsegment 85th Percentile Speed

The 85th percentile speed represents the speed of the vehicle whose speed is exceeded by only 15% of the population of vehicles. The speed of interest is that of vehicles traveling along the street approaching the subject intersection. It is measured at a location sufficiently distant from the intersection that speed is not influenced by intersection operation. This speed is likely to be influenced by traffic conditions, so it should reflect the conditions present during the analysis period.

Pedestrian Flow Rate

The pedestrian flow rate represents the count of pedestrians traveling through each corner of the intersection divided by the analysis period duration. It is expressed as an hourly flow rate but may represent an analysis period shorter than 1 h. This flow rate is provided for each of five movements at each intersection corner. These five movements (i.e., v_{ci}, v_{co}, v_{di}, v_{do}, and $v_{a,b}$) are shown in Exhibit 19-32 as they occur at one intersection corner.

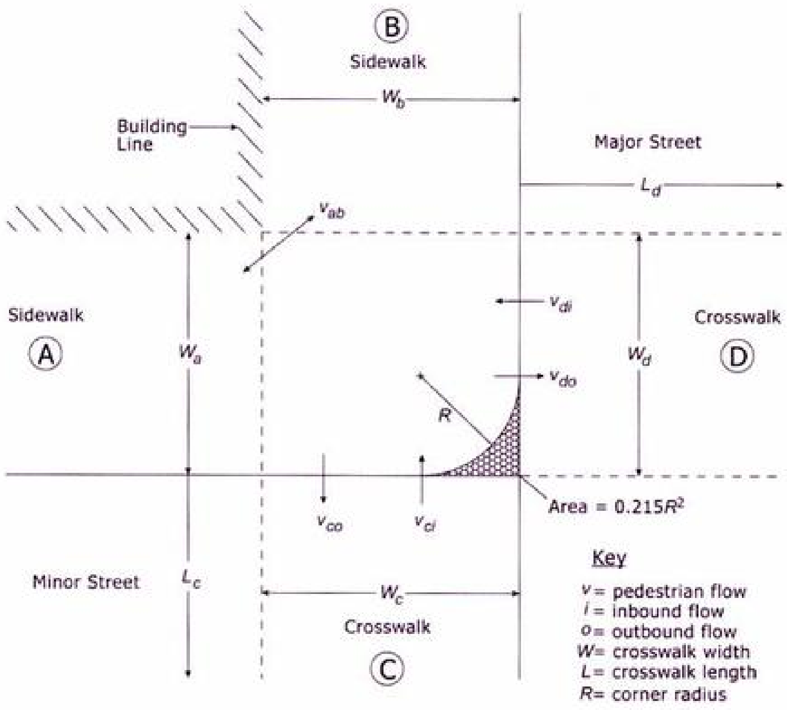
Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 19-31. These data describe the geometric elements that influence intersection performance from a pedestrian perspective. The number-of-lanes variable is defined in Section 3 for the motorized vehicle mode.

Number of Right-Turn Islands

The number of right-turn islands represents the count of channelizing islands encountered by pedestrians while crossing one intersection leg. The island should be delineated by a raised curb and of sufficient size to be considered a refuge for pedestrians. The number provided must have a value of 0, 1, or 2.

Exhibit 19-32
Intersection Corner Geometry and Pedestrian Movements



Total Walkway Width, Crosswalk Width and Length, and Corner Radius

The geometric design data of total walkway width, crosswalk width and length, and corner radius describe the pedestrian accommodations on each corner of the intersection. These data are shown in Exhibit 19-32. The total walkway width (i.e., W_a and W_b) is measured from the outside edge of the road pavement (or face of curb, if present) to the far edge of the sidewalk (as sometimes delineated by a building face, fence, or landscaping).

The crosswalk width (i.e., W_c and W_d) represents an effective width. Unless there is a known width constraint, the crosswalk's effective width should be the same as its physical width. A width constraint may be found when vehicles are observed to encroach regularly into the crosswalk area or when an obstruction in the median (e.g., a signal pole or reduced-width cut in the median curb) narrows the walking space.

The crosswalk length (i.e., L_c and L_d) is measured from outside edge to outside edge of road pavement (or curb to curb, if present) along the marked pedestrian travel path.

Signal Control Data

This subsection describes the data in Exhibit 19-31 that are identified as signal control. The walk, pedestrian clear, yellow change, and red clearance settings are defined in Section 3 for the motorized vehicle mode.

Rest in Walk

A phase with the rest-in-walk mode enabled will dwell in walk as long as there are no conflicting calls. When a conflicting call is received, the pedestrian clear interval will time to its setting value before ending the phase. This mode can be enabled for any actuated phase. Signals that operate with coordinated-actuated operation may be set to use a coordination mode that enables the rest-in-walk mode. Typically, the rest-in-walk mode is not enabled. In this case, the walk and pedestrian clear intervals time to their respective setting values, and the pedestrian signal indication dwells in a steady DON'T WALK indication until a conflicting call is received.

Cycle Length

Cycle length is predetermined for pretimed or coordinated-actuated control. Chapter 31 provides a procedure for estimating a reasonable cycle length for these two types of control when cycle length is unknown. Default values for cycle length are defined in Section 3 of the present chapter for the motorized vehicle mode.

For semiactuated and fully actuated control, an average cycle length must be provided as input to use the pedestrian or bicycle methodologies. This length can be estimated by using the motorized vehicle methodology.

Duration of Phase Serving Pedestrians

The duration of each phase that serves a pedestrian movement is a required input. This phase is typically the phase that serves the through movement that is adjacent to the sidewalk and for which the pedestrian, bicycle, and through vehicle travel paths are parallel. For example, Phases 2, 4, 6, and 8 are the phases serving the pedestrian movements in Exhibit 19-2.

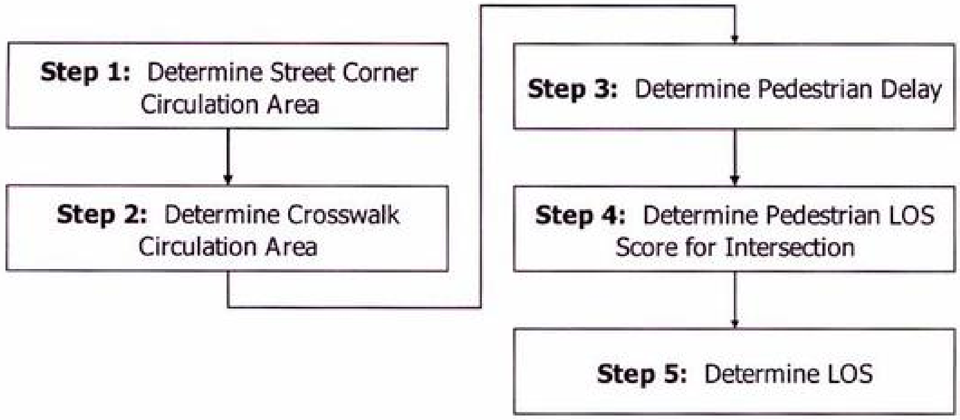
Pedestrian Signal Head Presence

The presence of a pedestrian signal head influences pedestrian crossing behavior. If a pedestrian signal head is provided, then pedestrians are assumed to use the crosswalk during the WALK and flashing DON'T WALK indications. If no pedestrian signal heads are provided, then pedestrians will cross during the green indication provided to vehicular traffic.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to pedestrians. The methodology is applied through a series of five steps that determine the pedestrian LOS for a crosswalk and associated corners. These steps are illustrated in Exhibit 19-33.

Exhibit 19-33
Pedestrian Methodology for Signalized Intersections



The methodology is focused on the analysis of signalized intersection performance. Chapter 18, Urban Street Segments, and Chapter 20, Two-Way STOP-Controlled Intersections, describe methodologies for evaluating the performance of these system elements with respect to the pedestrian mode.

COMPUTATIONAL STEPS

Step 1: Determine 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 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 19-51 is used to compute the time–space available at an intersection corner. Exhibit 19-32 identifies the variables used in the equation.

$$TS_{\text{corner}} = C(W_a\, W_b - 0.215\, R^2)$$

Equation 19-51

where

TS_{corner} = available corner time–space (ft^2-s),

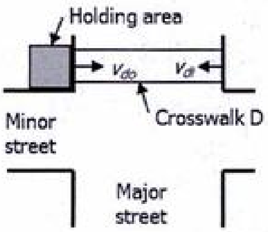
C = cycle length (s),

W_a = total walkway width of Sidewalk A (ft),

W_b = total walkway width of Sidewalk B (ft), and

R = radius of corner curb (ft).

If the corner curb radius is larger than either W_a or W_b, then the variable R in Equation 19-51 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.

Condition 1: Minor-Street Crossing

For Condition 1 (shown in Exhibit 19-29), Equation 19-52, with Equation 19-53, is used to compute holding-area time for pedestrians waiting to cross the major street.

Equation 19-52

$$Q_{tdo} = \frac{N_{do}\left(C - g_{\text{Walk},mi}\right)^2}{2\,C}$$

with

Equation 19-53

$$N_{do} = \frac{v_{do}}{3{,}600}C$$

where

Q_{tdo} = total time spent by pedestrians waiting to cross the major street during one cycle (p-s),

N_{do} = number of pedestrians arriving at the corner during each cycle to cross the major street (p),

$g_{\text{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s),

C = cycle length (s), and

v_{do} = flow rate of pedestrians arriving at the corner to cross the major street (p/h).

Research indicates pedestrians typically continue to enter intersections with pedestrian signal heads during the first few seconds of the pedestrian clear interval (*26, 32*). 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 some pedestrians are initiating their crossing during the flashing DON'T WALK indication.

The following guidance is provided to estimate the effective walk time on the basis of these research findings (*26, 32*). If the phase providing service to the pedestrians is either (*a*) actuated with a pedestrian signal head and rest in walk is *not* enabled or (*b*) pretimed with a pedestrian signal head, then Equation 19-54 is used.

Equation 19-54

$$g_{\text{Walk},mi} = Walk_{mi} + 4.0$$

If the phase providing service to the pedestrians is actuated with a pedestrian signal head and rest in walk is enabled, then Equation 19-55 is used.

Equation 19-55

$$g_{\text{Walk},mi} = D_{p,mi} - Y_{mi} - R_{c,mi} - PC_{mi} + 4.0$$

If otherwise (i.e., there is no pedestrian signal head), Equation 19-56 is used.

Equation 19-56

$$g_{\text{Walk},mi} = D_{p,mi} - Y_{mi} - R_{c,mi}$$

where

$g_{Walk,mi}$ = 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

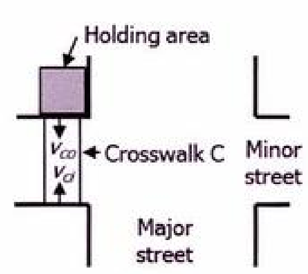
$R_{c,mi}$ = red clearance interval of the phase serving the minor-street through movement (s).

The effective walk time estimated with Equation 19-54 or Equation 19-55 can vary widely among intersections (*26, 32*). At a given intersection, the additional walk time can vary from 0.0 s to an amount equal to the pedestrian clear interval. The amount of additional walk time used by pedestrians depends on many factors, including the extent of pedestrian delay, vehicular volume, level of enforcement, and the presence of countdown pedestrian signal heads.

The effective walk time estimated with Equation 19-54 or Equation 19-55 is considered to be directly applicable to design or planning analyses because it is conservative in the amount of additional walk time it includes. A larger value of effective walk time may be applicable to an operational analysis if (*a*) field observation or experience indicates such a value would be consistent with actual pedestrian use of the flashing DON'T WALK indication; (*b*) an accurate estimate of pedestrian delay or queue size is desired; or (*c*) the predicted performance estimates are understood to reflect some illegal pedestrian behavior, possibly in response to constrained spaces or inadequate signal timing.

Condition 2: Major-Street Crossing

For Condition 2, as shown in Exhibit 19-30, the previous equations are repeated to compute the holding-area time for pedestrians waiting to cross the minor street Q_{tco}. For this application, the subscript letters *do* are replaced with the letters *co* to denote the pedestrians arriving at the corner to cross in Crosswalk C. Similarly, the subscript letters *mi* are replaced with *mj* to denote signal-timing variables associated with the phase serving the major-street through movement.



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 19-57 is used to compute the time–space available for circulating pedestrians.

Equation 19-57

$$TS_c = TS_{\text{corner}} - [5.0\,(Q_{tdo} + Q_{tco})]$$

where TS_c is the time–space available for circulating pedestrians (ft²-s).

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 pedestrians consume walking through the corner area. The latter quantity equals the total circulation volume multiplied by the assumed average circulation time (= 4.0 s). Equation 19-58, with Equation 19-59, is used to compute corner circulation area.

Equation 19-58

$$M_{\text{corner}} = \frac{TS_c}{4.0\,N_{tot}}$$

with

Equation 19-59

$$N_{tot} = \frac{v_{ci} + v_{co} + v_{di} + v_{do} + v_{a,b}}{3{,}600}C$$

where

M_{corner} = corner circulation area per pedestrian (ft²/p),

N_{tot} = total number of circulating pedestrians who arrive each cycle (p),

v_{ci} = flow rate of pedestrians arriving at the corner after crossing the minor street (p/h),

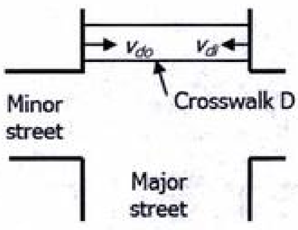
v_{co} = flow rate of pedestrians arriving at the corner to cross the minor street (p/h),

v_{di} = flow rate of pedestrians arriving at the corner after crossing the major street (p/h),

$v_{a,b}$ = flow rate of pedestrians traveling through the corner from Sidewalk A to Sidewalk B, or vice versa (p/h), and

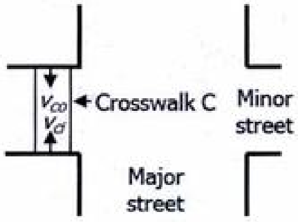
all other variables are as previously defined.

The circulation area obtained from Equation 19-58 can be compared with the ranges provided in Exhibit 19-28 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 that follows describes the evaluation of Crosswalk D in Exhibit 19-30 (i.e., a crosswalk across the major street). The procedure is repeated to evaluate Crosswalk C in Exhibit 19-29. 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 average pedestrian walking speed S_p is needed to evaluate corner and crosswalk performance. Research indicates 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.

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.

B. Compute Available Time–Space

Equation 19-60 is used to compute the time–space available in the crosswalk.

$$TS_{cw} = L_d\, W_d\, g_{\text{Walk},mi}$$

Equation 19-60

where

TS_{cw} = available crosswalk time–space (ft²-s),

L_d = length of Crosswalk D (ft),

W_d = effective width of Crosswalk D (ft), and

$g_{\text{Walk},mi}$ = effective walk time for the phase serving the minor-street through movement (s).

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 19-61 through Equation 19-63 are used for this purpose.

$$TS^*_{cw} = TS_{cw} - TS_{tv}$$

Equation 19-61

with

$$TS_{tv} = 40\, N_{tv}\, W_d$$

Equation 19-62

$$N_{tv} = \frac{v_{lt,perm} + v_{rt} - v_{rtor}}{3{,}600}\, C$$

Equation 19-63

where

TS^*_{cw} = effective available crosswalk time–space (ft²-s),

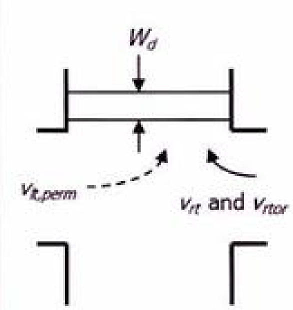
TS_{tv} = time–space occupied by turning vehicles (ft²-s),

N_{tv} = number of turning vehicles during the walk and pedestrian clear intervals (veh),

$v_{lt,perm}$ = permitted left-turn demand flow rate (veh/h),

v_{rt} = right-turn demand flow rate (veh/h), and

v_{rtor} = right-turn-on-red flow rate (veh/h).



The constant "40" in Equation 19-62 represents the product of the swept-path for most vehicles (= 8 ft) and the time a turning vehicle occupies the crosswalk (= 5 s). The left-turn and right-turn flow rates used in Equation 19-63 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 19-64 or Equation 19-65, depending on the crosswalk width, along with Equation 19-66. 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 (33).

If crosswalk width W_d is greater than 10 ft, then

Equation 19-64

$$t_{ps,do} = 3.2 + \frac{L_d}{S_p} + 2.7\frac{N_{ped,do}}{W_d}$$

If crosswalk width W_d is less than or equal to 10 ft, then

Equation 19-65

$$t_{ps,do} = 3.2 + \frac{L_d}{S_p} + 0.27N_{ped,do}$$

with

Equation 19-66

$$N_{ped,do} = N_{do}\frac{C - g_{\text{Walk},mi}}{C}$$

where

$t_{ps,do}$ = service time for pedestrians who arrive at the corner to cross the major street (s), and

$N_{ped,do}$ = number of pedestrians waiting at the corner to cross the major street (p).

Equation 19-66 estimates the number of pedestrians who cross as a group following the presentation of the WALK indication (or green indication, if pedestrian signal heads are not provided). It is also used to compute $N_{ped,di}$ for the other travel direction in the same crosswalk (using N_{di}, as defined below). Equation 19-64 or Equation 19-65 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 19-67 is used, with Equation 19-68 and results from previous steps, for the computation.

Equation 19-67

$$T_{occ} = t_{ps,do}N_{do} + t_{ps,di}N_{di}$$

with

Equation 19-68

$$N_{di} = \frac{v_{di}}{3{,}600}C$$

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).

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 19-69.

$$M_{cw} = \frac{TS^{*}_{cw}}{T_{occ}}$$

Equation 19-69

where M_{cw} is the crosswalk circulation area per pedestrian (ft²/p).

The circulation area obtained from Equation 19-69 can be compared with the ranges provided in Exhibit 19-28 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 19-30. The procedure is applied again to evaluate Crosswalk C shown in Exhibit 19-29. 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 19-70.

$$d_p = \frac{\left(C - g_{\mathrm{Walk},mi}\right)^2}{2\,C}$$

Equation 19-70

where d_p is pedestrian delay (s/p). The delay obtained from Equation 19-70 applies equally to both directions of travel along the crosswalk.

Research indicates average pedestrian delay at signalized intersection crossings is not constrained by capacity, even when pedestrian flow rates reach 5,000 p/h (*26*). For this reason, delay due to oversaturated conditions is not included in the value obtained from Equation 19-70.

If the subject crosswalk is closed, then the pedestrian delay d_p is estimated as the value obtained from Equation 19-70 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 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 (*34*). 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 that follows describes the evaluation of Crosswalk D in Exhibit 19-30. The procedure is repeated to evaluate Crosswalk C in Exhibit 19-29. For the second application, the subscript letter d is replaced with the letter c to denote the length and width of Crosswalk C. Also, the subscript letters mj are replaced with mi to denote variables associated with the minor street.

The pedestrian LOS score for the intersection $I_{p,int}$ is calculated by using Equation 19-71 through Equation 19-76.

Equation 19-71

$$I_{p,int} = 0.5997 + F_w + F_v + F_S + F_{\text{delay}}$$

with

Equation 19-72

$$F_w = 0.681(N_d)^{0.514}$$

Equation 19-73

$$F_v = 0.00569\left(\frac{v_{rtor} + v_{lt,perm}}{4}\right) - N_{rtci,d}\left(0.0027\, n_{15,mj} - 0.1946\right)$$

Equation 19-74

$$F_S = 0.00013\, n_{15,mj}\, S_{85,mj}$$

Equation 19-75

$$F_{\text{delay}} = 0.0401\, ln(d_{p,d})$$

Equation 19-76

$$n_{15,mj} = \frac{0.25}{N_d}\sum_{i \in m_d} v_i$$

where

$I_{p,int}$ = pedestrian LOS score for intersection,

F_w = cross-section adjustment factor,

F_v = motorized vehicle volume adjustment factor,

F_S = motorized vehicle speed adjustment factor,

F_{delay} = pedestrian delay adjustment factor,

$\ln(x)$ = natural logarithm of x,

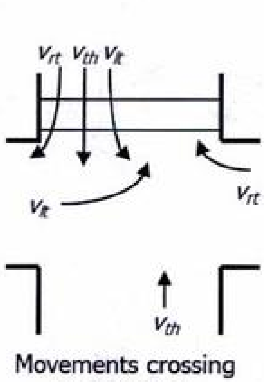
N_d = number of traffic lanes crossed when traversing Crosswalk D (ln),

$N_{rtci,d}$ = number of right-turn channelizing islands along Crosswalk D,

$n_{15,mj}$ = count of vehicles traveling on the major street during a 15-min period (veh/ln),

v_i = demand flow rate for movement i (veh/h),

m_d = set of all motorized vehicle movements that cross Crosswalk D (see figure in margin),

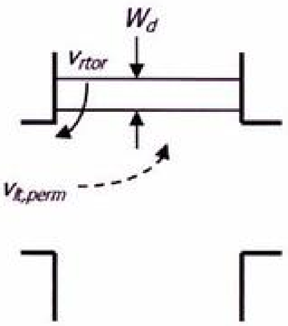


Movements crossing Crosswalk D

$S_{85,mj}$ = 85th percentile speed at a midsegment location on the major street (mi/h), and

$d_{p,d}$ = pedestrian delay when traversing Crosswalk D (s/p).

The left-turn flow rate $v_{lt,perm}$ used in Equation 19-73 is the flow rate associated with the left-turn movement that receives a green indication concurrently with the subject pedestrian crossing *and* turns across the subject crosswalk. The RTOR flow rate v_{rtor} is the flow rate associated with the approach being crossed and that also turns across the subject crosswalk. It is not the same v_{rtor} used in Equation 19-63.



The pedestrian LOS score obtained from this equation applies equally to both directions of travel along the crosswalk.

N_{rtci}, the variable for number of right-turn channelizing islands, is an integer with a value of 0, 1, or 2.

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 19-9 to determine the LOS for the subject crosswalk.

6. BICYCLE METHODOLOGY

This section describes the methodology for evaluating the quality of service provided to bicyclists traveling through a signalized intersection.

SCOPE OF THE METHODOLOGY

The overall scope of the three methodologies was provided in Section 2. This section identifies the additional conditions applicable to the bicycle methodology.

- *Target travel modes.* The bicycle methodology addresses travel by bicycle through a signalized intersection. It is not designed to evaluate the performance of other travel means (e.g., motorized bicycle, rickshaw).
- *Shared or exclusive lanes.* The bicycle methodology can be used to evaluate the service provided to bicyclists when sharing a lane with motorized vehicles or when traveling in an exclusive bicycle lane.

Spatial Limits

Intersection performance is evaluated separately for each intersection approach. *Unless otherwise stated, all variables identified in this subsection are specific to one intersection approach.* The bicycle is assumed to travel in the street (possibly in a bicycle lane) and in the same direction as adjacent motorized vehicles.

Performance Measures

Performance measures applicable to the bicycle travel mode include bicycle delay and bicycle LOS score. The LOS score is an indication of the typical bicyclist's perception of the overall crossing experience.

LOS is also considered a performance measure. It is useful for describing intersection performance to elected officials, policy makers, administrators, or the public. LOS is based on the bicyclist LOS score.

Limitations of the Methodology

This subsection identifies the known limitations of the bicycle methodology. If one or more of these limitations is believed to have an important influence on the performance of a specific intersection, then the analyst should consider using alternative methods or tools for the evaluation.

The bicycle methodology does not account for the effect of the following conditions on the quality of service provided to bicyclists:

- Presence of grades in excess of 2% and
- Presence of railroad crossings.

REQUIRED DATA AND SOURCES

This subsection describes the input data needed for the bicycle methodology. These data are listed in Exhibit 19-34. The second column (labeled Basis) of the exhibit indicates whether the input data are needed for each signal phase, each intersection approach, or the intersection as a whole. The exhibit also lists default values that can be used if local data are not available (*25, 30*).

Exhibit 19-34
Required Input Data, Potential Data Sources, and Default Values for Bicycle Analysis

Required Data and Units	Basis	Potential Data Source(s)	Suggested Default Value
Traffic Characteristics			
Motorized vehicle demand flow rate (veh/h)	A	Field data, past counts	Must be provided
Bicycle flow rate (bicycles/h)	A	Field data, past counts	Must be provided
Proportion of on-street parking occupied (decimal)	A	Field data	0.50 (if parking lane present)
Geometric Design			
Street width (ft)	A	Field data, aerial photo	Based on a 12-ft lane width
Number of lanes	A	Field data, aerial photo	Must be provided
Width of outside through lane (ft)	A	Field data, aerial photo	12 ft
Width of bicycle lane (ft)	A	Field data, aerial photo	5.0 ft (if provided)
Width of paved outside shoulder (ft)	A	Field data, aerial photo	Must be provided
Width of striped parking lane (ft)	A	Field data, aerial photo	8.0 ft (if present)
Signal Control Data			
Cycle length (s)	I	Field data	Same as motorized vehicle mode
Yellow change + red clearance (s)[a]	P	Field data	4 s
Duration of phase serving bicycles (s)	P	Field data	Same as motorized vehicle mode
Other Data			
Analysis period duration (h)[b]	I	Set by analyst	0.25 h

Notes: A = approach: one value for the intersection approach.
I = intersection: one value for the intersection.
P = phase: one value or condition for each signal phase.
[a] Specific values of yellow change and red clearance should be determined by local guidelines or practice.
[b] Analysis period duration is as defined for Exhibit 19-11.

The data elements listed in Exhibit 19-34 do not include variables that are considered to represent calibration factors. Default values are provided for these factors because they typically have a relatively narrow range of reasonable values or they have a small impact on the accuracy of the performance estimates. The recommended value for each calibration factor is identified at the relevant point during presentation of the methodology.

Traffic Characteristics Data

This subsection describes the traffic characteristics data listed in Exhibit 19-34. These data describe the traffic streams traveling through the intersection during the study period. The demand flow rate of motorized vehicles and bicycle flow rate are defined in Section 3 for the motorized vehicle mode.

The variable for the proportion of on-street parking occupied represents the proportion of the intersection's right-side curb line that has parked vehicles present during the analysis period. It is based on a zone that extends from a point 250 ft upstream of the intersection to the intersection and a second zone that extends from the intersection to a point 250 ft downstream of the intersection. If parking is not allowed in these two zones, then this proportion equals 0.0.

Geometric Design Data

This subsection describes the geometric design data listed in Exhibit 19-34. These data describe the geometric elements that influence intersection performance from a bicyclist perspective. The number-of-lanes variable is defined in Section 3 for the motorized vehicle mode.

Street Width

The street width represents the width of the cross street as measured along the outside through vehicle lane on the subject approach between the extended curb line limits of the cross street. It is measured for each intersection approach.

Widths of Outside Through Lane, Bicycle Lane, Outside Shoulder, and Parking Lane

The widths of several individual elements of the cross section are considered input data. These elements include the outside lane that serves motorized vehicles at the intersection, the bicycle lane adjacent to the outside lane (if used), paved outside shoulder, and striped parking lane.

The outside lane width does not include the width of the gutter. If curb and gutter are present, then the width of the gutter is included in the shoulder width (i.e., shoulder width is measured to the curb face when a curb is present).

Signal Control Data

This subsection describes the data in Exhibit 19-34 that are identified as signal control. The yellow change interval and red clearance interval settings are defined in Section 3 for the motorized vehicle mode.

Cycle Length

Cycle length is predetermined for pretimed or coordinated-actuated control. Chapter 31 provides a procedure for estimating a reasonable cycle length for these two types of control when cycle length is unknown. Default values for cycle length are defined in Section 3 of the present chapter for the motorized vehicle mode.

For semiactuated and fully actuated control, an average cycle length must be provided as input to use the pedestrian or bicycle methodologies. This length can be estimated by using the motorized vehicle methodology.

Duration of Phase Serving Pedestrians and Bicycles

The duration of each phase that serves a bicycle movement is a required input. This phase is typically the phase that serves the through movement that is adjacent to the sidewalk and for which the bicycle and through vehicle travel paths are parallel. For example, Phases 2, 4, 6, and 8 are the phases serving the bicycle movements in Exhibit 19-2.

OVERVIEW OF THE METHODOLOGY

This subsection describes the methodology for evaluating the performance of a signalized intersection in terms of its service to bicyclists. The methodology is applied through a series of three steps that determine the bicycle LOS for an intersection approach. These steps are illustrated in Exhibit 19-35.

The methodology is focused on analyzing signalized intersection performance from the bicyclist point of view. Chapter 18, Urban Street Segments, describes a methodology for evaluating urban street performance with respect to the bicycle mode.

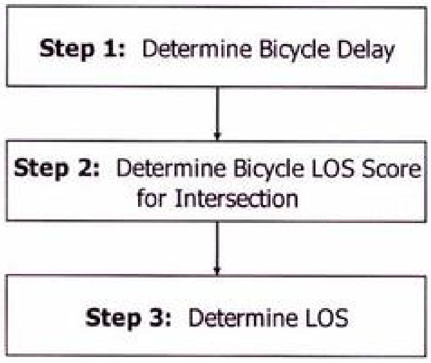


Exhibit 19-35
Bicycle Methodology for Signalized Intersections

COMPUTATIONAL STEPS

Step 1: Determine Bicycle Delay

This step describes a procedure for evaluating the performance of one intersection approach. It is repeated for each approach of interest.

Bicycle delay can be calculated only for intersection approaches that have an on-street bicycle lane or a shoulder that can be used by bicyclists as a bicycle lane. Bicyclists who share a lane with motorized vehicle traffic will incur the same delay as the motorized vehicles.

A. Compute Bicycle Lane Capacity

A wide range of capacities and saturation flow rates have been reported by many countries for bicycle lanes at intersections. Research indicates the base saturation flow rate may be as high as 2,600 bicycles/h (*35*). However, few intersections provide base conditions for bicyclists, and current information is insufficient to calibrate a series of appropriate saturation flow adjustment factors. Until such factors are developed, it is recommended that a saturation flow rate of 2,000 bicycles/h be used as an average value achievable at most intersections.

A saturation flow rate of 2,000 bicycles/h assumes right-turning motor vehicles yield the right-of-way to through bicyclists. Where aggressive right-turning traffic exists, 2,000 bicycles/h may not be achievable. Local observations to determine a saturation flow rate are recommended in such cases.

The capacity of the bicycle lane at a signalized intersection may be computed with Equation 19-77.

$$c_b = s_b \frac{g_b}{C}$$

Equation 19-77

where

c_b = capacity of the bicycle lane (bicycles/h),

s_b = saturation flow rate of the bicycle lane = 2,000 (bicycles/h),

g_b = effective green time for the bicycle lane (s), and

C = cycle length (s).

The effective green time for the bicycle lane can be assumed to equal that for the adjacent motor vehicle traffic stream that is served concurrently with the subject bicycle lane (i.e., $g_b = D_p - l_1 - l_2$).

B. Compute Bicycle Delay

Bicycle delay is computed with Equation 19-78.

Equation 19-78

$$d_b = \frac{0.5\,C\,(1 - g_b/C)^2}{1 - \min\left(\frac{v_{bic}}{c_b}, 1.0\right)\frac{g_b}{C}}$$

where d_b is bicycle delay (s/bicycle), v_{bic} is bicycle flow rate (bicycles/h), and all other variables are as previously defined.

This delay equation is based on the assumption there is no bicycle incremental delay or initial queue delay. Bicyclists will not normally tolerate an oversaturated condition and will select other routes or ignore traffic regulations to avoid the associated delays.

At most signalized intersections, the only delay to through bicycles is caused by the signal, because bicycles have the right-of-way over right-turning vehicles during the green indication. Bicycle delay could be longer than that obtained from Equation 19-78 when (*a*) bicycles are forced to weave with right-turning traffic during the green indication or (*b*) drivers do not acknowledge the bicycle right-of-way because of high flows of right-turning vehicles.

The delay obtained from Equation 19-78 can be used to make some judgment about intersection performance. Bicyclists tend to have about the same tolerance for delay as pedestrians. They tend to become impatient when they experience a delay in excess of 30 s/bicycle. In contrast, they are very likely to comply with the signal indication if their expected delay is less than 10 s/bicycle.

Step 2: Determine Bicycle LOS Score for Intersection

This step describes a procedure for evaluating the performance of one intersection approach. It is repeated for each approach of interest. The bicycle LOS score can be calculated for any intersection approach, regardless of whether it has an on-street bicycle lane.

The bicycle LOS score for the intersection $I_{b,int}$ is calculated by using Equation 19-79 through Equation 19-82.

Equation 19-79

$$I_{b,int} = 4.1324 + F_w + F_v$$

with

Equation 19-80

$$F_w = 0.0153\,W_{cd} - 0.2144\,W_t$$

Equation 19-81

$$F_v = 0.0066\frac{v_{lt} + v_{th} + v_{rt}}{4\,N_{th}}$$

Equation 19-82

$$W_t = W_{ol} + W_{bl} + W_{os}^{*} + I_{pk}W_{pk}$$

where

$I_{b,int}$ = bicycle LOS score for intersection;

F_w = cross-section adjustment factor;

F_v = motorized vehicle volume adjustment factor;

W_{cd} = curb-to-curb width of the cross street (ft);

W_t = total width of the outside through lane, bicycle lane, and paved shoulder (ft);

v_{lt} = left-turn demand flow rate (veh/h);

v_{th} = through demand flow rate (veh/h);

v_{rt} = right-turn demand flow rate (veh/h);

N_{th} = number of through lanes (shared or exclusive) (ln);

W_{ol} = width of the outside through lane (ft);

W_{bl} = width of the bicycle lane (= 0.0 if bicycle lane not provided) (ft);

W_{pk} = width of striped parking lane (ft);

I_{pk} = indicator variable for on-street parking occupancy (= 0 if $p_{pk} > 0.0$, 1 otherwise);

p_{pk} = proportion of on-street parking occupied (decimal);

W_{os} = width of paved outside shoulder (ft); and

W_{os}^* = adjusted width of paved outside shoulder (if curb is present, $W_{os}^* = W_{os} - 1.5 \geq 0.0$; otherwise, $W_{os}^* = W_{os}$) (ft).

Step 3: Determine LOS

This step describes a process for determining the LOS of one intersection approach. It is repeated for each approach of interest.

The bicycle LOS is determined by using the bicycle LOS score from Step 2. This performance measure is compared with the thresholds in Exhibit 19-9 to determine the LOS for the subject approach.

7. APPLICATIONS

EXAMPLE PROBLEMS

Chapter 31, Signalized Intersections: Supplemental, describes the application of each of the three methodologies through the use of example problems. There is one example problem associated with each methodology. The examples illustrate the operational analysis type.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 19-36 shows an illustrative generalized service volume table for a signalized intersection. This particular exhibit has been prepared for illustrative purposes only and should not be used for any specific planning or preliminary engineering application because the values in the table are highly dependent on the assumed input variables. Care must be taken in constructing a table that the analyst believes is representative of a typical signalized intersection within the planning area. In the example table, the volumes represent the total approach volume (sum of the left-, through, and right-turn movements). This particular table illustrates how hourly service volumes vary with the number of through lanes on the approach and the through movement *g*/*C* ratio.

Exhibit 19-36
Illustrative Generalized Service Volumes for Signalized Intersections (veh/h)

Through Movement *g/C* Ratio	No. of Through Lanes	LOS B	LOS C	LOS D	LOS E
0.40	1	130	610	730	800
	2	270	1,220	1,430	1,550
	3	380	1,620	1,980	2,000
0.45	1	320	720	840	910
	2	630	1,410	1,610	1,740
	3	840	1,780	2,000	2,250
0.5	1	490	830	940	1,020
	2	940	1,580	1,790	1,930
	3	1,180	1,930	2,000	2,500

Notes: LOS E threshold is defined by control delay greater than 80 s/veh or volume-to-capacity ratio >1.0.
Assumed values for all entries:
- Heavy vehicles: 0%
- Peak hour factor: 0.92
- Lane width: 12 ft
- Grade: 0%
- Separate left-turn lane: yes
- Separate right-turn lane: no
- Pretimed control
- Cycle length: 90 s
- Lost time: 4 s/phase
- Protected left-turn phasing: yes
- g/C ratio for left-turn movement: 0.10
- Parking maneuvers per hour: 0
- Buses stopping per hour: 0
- Percentage left turns: 10%
- Percentage right turns: 10%

The hourly service volumes could easily be converted to daily service volumes with the application of appropriate *K*- and *D*-factors. Step-by-step instructions are provided in Appendix B of Chapter 6, HCM and Alternative Analysis Tools, for users wishing to learn more about constructing one's own service volume table.

ANALYSIS TYPE

The three methodologies described in this chapter can each be used in three types of analysis. These analysis types are described as operational, design, and planning and preliminary engineering. The characteristics of each analysis type are described in the subsequent paragraphs.

Operational Analysis

The objective of an operational analysis is to determine the LOS for current or near-term conditions when details of traffic volumes, geometry, and traffic control conditions are known. All the methodology steps are implemented and all calculation procedures are applied for the purpose of computing a wide range of performance measures. The operational analysis type will provide the most reliable results because it uses no (or minimal) default values.

Design Analysis

The objective of a design analysis is to identify the alternatives that operate at the target level of the specified performance measures (or provide a better level of performance). The analyst may then recommend the "best" design alternative after consideration of the full range of factors.

The design analysis type has two variations. Both variations require specifying the traffic conditions and target levels for a set of performance measures. One variation requires the additional specification of the signalization conditions. The methodology is then applied by using an iterative approach in which alternative geometric conditions are separately evaluated.

The second variation of the design analysis requires the additional specification of geometric conditions. The methodology is then applied by using an iterative approach in which alternative signalization conditions are evaluated.

Planning and Preliminary Engineering Analysis

The objective of a planning and preliminary engineering analysis can be (*a*) to determine the LOS for either a proposed intersection or an existing intersection in a future year or (*b*) to size the overall geometrics of a proposed intersection.

The level of precision inherent in planning and preliminary engineering analyses is typically lower than for operational analyses because default values are often substituted for field-measured values of many of the input variables. Recommended default values for this purpose are described above in the sections associated with each methodology.

The requirement for a complete description of the signal-timing plan can be a burden for some planning analyses. The planning-level analysis application described in Chapter 31, Signalized Intersections: Supplemental, can be used to estimate a reasonable timing plan, in conjunction with the default values provided.

General alternative tool guidance is provided in Chapters 6 and 7.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools, and Chapter 7, Interpreting HCM and Alternative Tool Results. This subsection contains specific guidance for the application of alternative tools to the analysis of signalized intersections. Additional information on this topic is provided in the Technical Reference Library in Volume 4. The focus of this subsection is the application of alternative tools to evaluate motorized vehicle operation.

Comparison of Motorized Vehicle Methodology and Alternative Tools

Motorized Vehicle Methodology

The motorized vehicle methodology models the driver–vehicle–road–signal system with reasonable accuracy for most applications. It accounts for the operation of actuated phases, shared lanes, and permitted turn movements. It can account for the effect of initial queues and signal progression on delay.

The motorized vehicle methodology offers several advantages over alternative analysis tools. One advantage is that it has an empirically calibrated procedure for estimating saturation flow rate. Alternative tools often require saturation flow rate as an input variable. A second advantage is that it produces a direct estimate of capacity and volume-to-capacity ratio. These measures are not directly available from simulation tools. A third advantage is that it produces an expected value for each of several performance measures in a single application. Simulation tools require multiple runs and manual calculations to obtain an expected value for a given performance measure. A fourth advantage is that its analytic procedures are described in the HCM so that analysts can understand the driver–vehicle–road interactions and the means by which they are modeled. Most proprietary alternative tools operate as a "black box," providing little detail describing the intermediate calculations.

Alternative Tools

Deterministic tools and simulation tools are in common use as alternatives to the motorized vehicle methodology offered in this chapter. Deterministic tools are often used for the analysis of signalized intersections. The main reasons for their popularity are found in the user interface, optimization options, and output presentation features. Some also offer additional performance measures such as fuel consumption, air quality, and operating cost.

Conceptual Differences

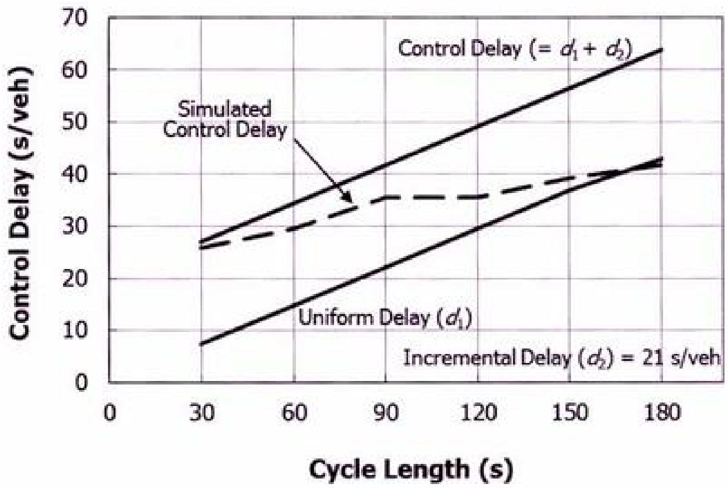
Conceptual differences in modeling approach may preclude the direct comparison of performance measures from the motorized vehicle methodology with those from alternative tools. The treatment of random arrivals is a case in point. There is a common misconception among analysts that alternative tools treat random arrivals in a similar manner.

A simple case is used to demonstrate the different ways alternative tools model random arrivals. Consider an isolated intersection with a two-phase sequence. The subject intersection approach serves only a through movement;

there are no turning movements from upstream intersections or driveways. The only parameter that is allowed to vary in this example is the cycle length (all other variables are held constant).

The results of this experiment are shown in Exhibit 19-37. The two solid lines represent delay estimates obtained from the motorized vehicle methodology. Uniform delay is shown to increase linearly with cycle length. Incremental delay is constant with respect to cycle length because the volume-to-capacity ratio is constant. As a result, control delay (the sum of the uniform delay and incremental delay) is also shown to increase linearly with cycle length.

Exhibit 19-37
Effect of Cycle Length on Delay



The dashed line in Exhibit 19-37 represents the control delay estimate obtained from a simulation-based analysis tool. The simulation-based tool shows close agreement with the motorized vehicle methodology for short cycles, but it deviates for longer cycles. There are likely to be explainable reasons for this difference; however, the point is that such differences are likely to exist among tools. The analyst should understand the underlying modeling assumptions and limitations inherent in any tool (including the motorized vehicle methodology) when it is used. Moreover, the analyst should fully understand the definition of any performance measure used so as to interpret the results and observed trends properly.

Alternative Tool Application Guidance

Development of HCM-Compatible Performance Measures

The motorized vehicle methodology is used to predict control delay, which is defined as the excess travel time caused by the action of the control device (in this case, the signal). Simulation-based analysis tools often use a definition of delay that differs from that used in the motorized vehicle methodology, especially for movements that are oversaturated at some point during the analysis. Therefore, some care must be taken in the determination of LOS when simulation-based delay estimates are used. Delay comparison among different tools is discussed in more detail in Chapter 7.

An accurate estimate of control delay may be obtained from a simulation tool by performing simulation runs with and without the control device(s) in place. The segment delay reported with no control is the delay due to geometrics and interaction between vehicles. The additional delay reported in the run with the control in place is, by definition, the control delay.

Adjustment of Parameters

For applications in which either an alternative tool or the motorized vehicle methodology can be used, some adjustment will generally be required for the alternative tool if some consistency with the motorized vehicle methodology is desired. For example, the parameters that determine the capacity of a signalized approach (e.g., saturation flow rate and start-up lost time) should be adjusted to ensure the lane group (or approach) capacities from the alternative tool match those estimated by the motorized vehicle methodology.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 31, Signalized Intersections: Supplemental, includes examples to illustrate the use of simulation tools to address the stated limitations of this chapter. Specifically, these examples address the following conditions: left-turn storage-bay overflow, RTOR operation, short through lanes, and closely spaced intersections.

8. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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CHAPTER 20
TWO-WAY STOP-CONTROLLED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Two-way STOP-controlled (TWSC) intersections are common in the United States. One typical configuration is a four-leg intersection in which one street—the *major street*—is uncontrolled, and the other street—the *minor street*—is controlled by STOP signs. The other typical configuration is a three-leg intersection in which the single minor-street approach (i.e., the stem of the T configuration) is controlled by a STOP sign. Minor-street approaches can be public streets or private driveways. This chapter presents concepts and procedures for analyzing these types of intersections. Chapter 9 provides a glossary and list of symbols, including those used for TWSC intersections.

Three-leg intersections in which the stem of the T is controlled by a STOP sign are considered a standard type of TWSC intersection.

Capacity analysis of TWSC intersections requires a clear description and understanding of the interaction between travelers on the minor (i.e., STOP-controlled) approach with travelers on the major street. Both analytical and regression models have been developed to describe this interaction. Procedures described in this chapter rely primarily on field measurements of TWSC performance in the United States (*1*) that have been applied to a gap acceptance model developed and refined in Germany (*2*).

CHAPTER ORGANIZATION

This chapter is organized into the following sections:

- Section 1 (this section) introduces the chapter.
- Section 2 describes the basic concepts of the TWSC procedure. Most notably, the concept of gap acceptance—which is the basis of TWSC intersection operations—is described. Performance measures and level-of-service (LOS) criteria are also discussed.
- Section 3 provides the details of the TWSC intersection analysis procedure for the motorized vehicle mode, including required input data and detailed computational steps.
- Section 4 extends the motorized vehicle mode procedure to account for the effects of pedestrians on capacity.
- Section 5 presents a procedure for analyzing pedestrian operations at a TWSC intersection, including required data and computational steps.
- Section 6 qualitatively discusses bicycle operations at a TWSC intersection and directs the reader to related research.
- Section 7 describes example problems included in Volume 4, suggests applications for alternative tools, and provides guidance on interpreting analysis results.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 4, Traffic Operations and Capacity Concepts, introduces concepts of traffic flow and capacity that apply to TWSC intersections, including peak hour factor, gap acceptance, and control delay.
- Chapter 5, Quality and Level-of-Service Concepts, provides an overview of the LOS concept used throughout the HCM.
- Chapter 32, STOP-Controlled Intersections: Supplemental, provides example problems demonstrating the TWSC methodology.
- Case Study 1, U.S. 95 Corridor, and Case Study 5, Museum Road, in the *HCM Applications Guide* in Volume 4, demonstrate how this chapter's methods can be applied to the evaluation of an actual TWSC intersection.
- Section M, STOP-Controlled Intersections, in the *Planning and Preliminary Engineering Applications Guide to the HCM* in Volume 4, provides guidance on analyzing TWSC intersections in the context of a planning study.

2. CONCEPTS

TWSC intersections are unsignalized intersections at which drivers on the major street have priority over drivers on the minor-street approaches. Minor-street drivers must stop before entering the intersection. Left-turning drivers from the major street must yield to oncoming major-street through or right-turning traffic, but they are not required to stop in the absence of oncoming traffic.

The methodologies presented rely on the required input data listed in Section 3 to compute the *potential capacity* of each minor movement, which is ultimately adjusted, if appropriate, to compute a *movement capacity* for each movement. The movement capacity can be used to estimate the control delay by movement, by approach, and for the intersection as a whole. Queue lengths can also be estimated once movement capacities are determined.

At TWSC intersections, drivers on the STOP-controlled approaches are required to select gaps in the major-street flow in order to execute crossing or turning maneuvers. In the presence of a queue, each driver on the controlled approach must also spend time moving to the front-of-queue position and prepare to evaluate gaps in the major-street flow. Thus, the capacity of the controlled legs is based primarily on three factors: the distribution of gaps in the major-street traffic stream, driver judgment in selecting gaps through which to execute the desired maneuvers, and the follow-up headways required by each driver in a queue.

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.

The basic capacity model assumes gaps in the conflicting movements are randomly distributed. When traffic signals are present on the major street upstream of the subject intersection, flows may not be random but will likely have some platoon structure.

When traffic signals are present on the major street upstream of the subject intersection, flows may not be random but will likely have some platoon structure.

For the analysis of the motorized vehicle mode, the methodology addresses special circumstances that may exist at TWSC intersections, including the following:

- Two-stage gap acceptance,
- Approaches with shared lanes,
- The presence of upstream traffic signals, and
- Flared approaches for minor-street right-turning vehicles.

INTERSECTION ANALYSIS BOUNDARIES AND TRAVEL MODES

The intersection boundaries for a TWSC intersection analysis are assumed to be those of an isolated intersection (i.e., not affected by upstream or downstream intersections), with the exception of TWSC intersections that may be affected by vehicle platoons from upstream signals. This chapter presents methodologies to assess TWSC intersections for both pedestrians and motor vehicles. A discussion of how the procedures for motor vehicles could potentially apply to an analysis of bicycle movements is also provided.

GAP ACCEPTANCE THEORY

Gap acceptance models begin with the recognition that TWSC intersections give no positive indication or control to the driver on the minor street as to when it is appropriate to leave the stop line and enter the major street. The driver must determine when a gap on the major street is large enough to permit entry and when to enter on the basis of the relative priority of the competing movements. This decision-making process has been formalized analytically into what is commonly known as gap acceptance theory. Gap acceptance theory includes three basic elements: the size and distribution (availability) of gaps on the major street, the usefulness of these gaps to the minor-street drivers, and the relative priority of the various movements at the intersection.

Availability of Gaps

The first element in gap acceptance theory is the proportion of gaps of a particular size on the major street offered to the driver entering from a minor movement, as well as the pattern of vehicle arrival times. The distribution of gaps between the vehicles in the different streams has a major effect on the performance of the intersection.

Usefulness of Gaps

The second element is the extent to which drivers find gaps of a particular size useful when they attempt to enter the intersection. It is generally assumed in gap acceptance theory that drivers are both consistent and homogeneous. This assumption is not entirely correct. Studies have demonstrated not only that drivers have different gap acceptance thresholds but that the gap acceptance threshold of an individual driver often changes over time (*3*). In this manual, the critical headways and follow-up headways are considered representative of a statistical average of the driver population in the United States.

Relative Priority of Various Movements at the Intersection

The third element in gap acceptance theory concerns the ranking of each movement in a priority hierarchy. Typically, gap acceptance processes assume drivers on the major street are unaffected by the minor movements. If this assumption is not the case at a given intersection, the gap acceptance process has to be modified.

In using the TWSC intersection methodology, the priority of right-of-way given to each movement must be identified. Some movements have absolute priority; other movements must yield to higher-order movements. Movements can be categorized by right-of-way priority as follows:

Pedestrian movements crossing the major street are assumed to be Rank 2 for the automobile analysis procedure. The effect of Rank 1 vehicles yielding to pedestrians is included in the pedestrian analysis procedure.

- Movements of Rank 1 include through traffic on the major street, right-turning traffic from the major street, and pedestrian movements crossing the minor street.
- Movements of Rank 2 (subordinate to Rank 1) include left-turning and U-turning traffic from the major street, right-turning traffic onto the major street, and pedestrian movements crossing the major street (assumed for this procedure).

- Movements of Rank 3 (subordinate to Ranks 1 and 2) include through traffic on the minor street (in the case of a four-leg intersection) and left-turning traffic from the minor street (in the case of a T-intersection).
- Movements of Rank 4 (subordinate to all others) include left-turning traffic from the minor street. Rank 4 movements occur only at four-leg intersections.

The minor-street left-turn movement is assigned Rank 3 priority at a T-intersection and Rank 4 priority at a four-leg intersection.

Exhibit 20-1 shows the assumed numbering of movements at both T- and four-leg intersections.

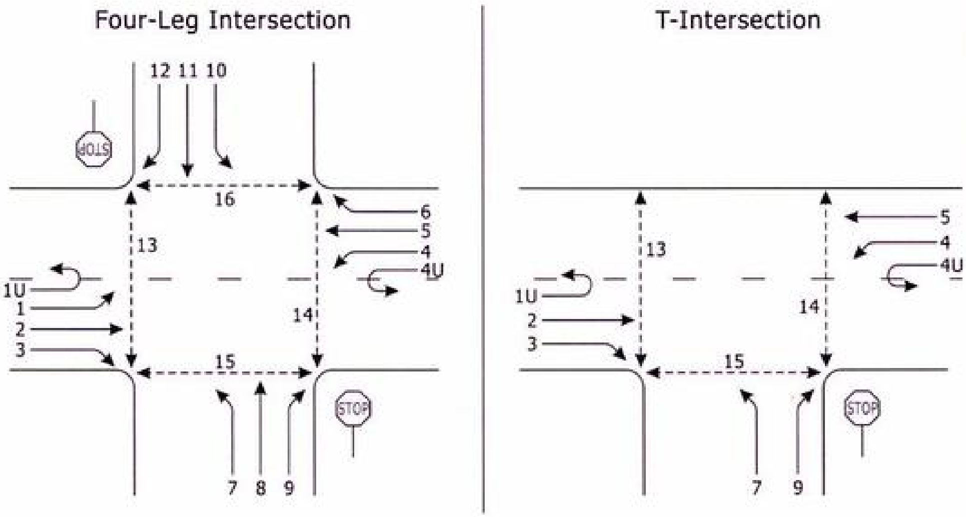


Exhibit 20-1
Vehicular and Pedestrian Movements at a TWSC Intersection

Solid arrows indicate vehicular movements; dashed arrows indicate pedestrian movements.

As an example of the application of right-of-way priority, assume the situation of a left-turning vehicle on the major street and a through vehicle from the minor street waiting to cross the major traffic stream. The first available gap of acceptable size would be taken by the left-turning vehicle. The minor-street through vehicle must wait for the second available gap. In aggregate terms, a large number of such left-turning vehicles could use up so many of the available gaps that minor-street through vehicles would be severely impeded or unable to make safe crossing movements.

Critical Headway and Follow-Up Headway

Critical headway defined.

Critical headway t_c is defined as the minimum time interval in the major-street traffic stream that allows intersection entry for one minor-street vehicle (*4*). Thus, the driver's critical headway is the minimum headway that would be acceptable. A particular driver would reject headways less than the critical headway and would accept headways greater than or equal to the critical headway. Critical headway can be estimated on the basis of observations of the largest rejected and smallest accepted headway for a given intersection.

Follow-up headway defined.

The time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major-street headway, under a condition of continuous queuing on the minor street, is called the *follow-up headway* t_f. Thus, t_f is the headway that defines the saturation flow rate for the approach if there were no conflicting vehicles on movements of higher rank.

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.

LOS for a TWSC intersection is determined by the computed or measured control delay. For motor vehicles, LOS is determined for each minor-street movement (or shared movement), as well as the major-street left turns, by using the criteria given in Exhibit 20-2. LOS is not defined for the intersection as a whole or for major-street approaches for three primary reasons: (*a*) major-street through vehicles are assumed to experience zero delay; (*b*) the disproportionate number of major-street through vehicles at a typical TWSC intersection skews the weighted average of all movements, resulting in a very low overall average delay for all vehicles; and (*c*) the resulting low delay can mask LOS deficiencies for minor movements. As Exhibit 20-2 notes, LOS F is assigned to a movement if its volume-to-capacity ratio exceeds 1.0, regardless of the control delay.

The LOS criteria for TWSC intersections differ somewhat from the criteria used in Chapter 19 for signalized intersections, primarily because user perceptions differ among transportation facility types. The expectation is that a signalized intersection is designed to carry higher traffic volumes and will present greater delay than an unsignalized intersection. Unsignalized intersections are also associated with more uncertainty for users, as delays are less predictable than they are at signals.

Exhibit 20-2
LOS Criteria: Motorized Vehicle Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio	
	$v/c \le 1.0$	$v/c > 1.0$
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 20-3.

Exhibit 20-3
LOS Criteria: Pedestrian Mode

LOS	Control Delay (s/p)	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 seconds per 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.

3. MOTORIZED VEHICLE CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

The version of the TWSC intersection analysis procedure presented in this section is primarily based on studies conducted by National Cooperative Highway Research Program Project 3-46 (*1*).

Spatial and Temporal Limits

This methodology assumes the TWSC intersection under investigation is isolated, with the exception of a TWSC intersection that may be affected by vehicle platoons from upstream signals. When interaction effects (e.g., queue spillback, demand starvation) are likely between the subject TWSC intersection and other intersections, the use of alternative tools may result in a more accurate analysis. Analysis boundaries may also include different demand scenarios related to the time of day or to different development scenarios that produce various demand flow rates.

The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This method produces the following performance measures:

- Volume-to-capacity ratio,
- Control delay,
- LOS based on control delay, and
- 95th percentile queue length.

Limitations of the Methodology

The methodologies in this chapter apply to TWSC intersections with up to three through lanes (either shared or exclusive) on the major-street approaches and up to three lanes on the minor-street approaches (with no more than one exclusive lane for each movement on the minor-street approach). Effects from other intersections are accounted for only in situations in which a TWSC intersection is located on an urban street segment between coordinated signalized intersections. In this situation, the intersection can be analyzed by using the procedures in Chapter 18, Urban Street Segments. The methodologies do not apply to TWSC intersections with more than four approaches or more than one STOP-controlled approach on each side of the major street.

The methodologies do not include a detailed method for estimating delay at YIELD-controlled intersections; however, with appropriate changes in the values of key parameters (e.g., critical headway and follow-up headway), the analyst could apply the TWSC method to YIELD-controlled intersections.

With appropriate changes in the values of critical headway and follow-up headway, the analyst could apply the TWSC method to YIELD-controlled intersections.

All the methods are for steady state conditions (i.e., the demand and capacity conditions are constant during the analysis period); the methods are not designed to evaluate how fast or how often the facility transitions from one demand or capacity state to another. Analysts interested in that kind of information should consider applying alternative tools, as discussed below.

Alternative Tool Considerations

Strengths of the HCM Procedure

This chapter offers a set of comprehensive procedures for analyzing the performance of an intersection under two-way STOP control. Simulation-based tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with the roadway and the control system, but for most purposes the HCM procedure produces an acceptable approximation.

The HCM procedure offers the advantage of a deterministic evaluation of a TWSC intersection, the results of which have been accepted by a broad consensus of international experts. The HCM procedure also considers advanced concepts such as two-stage gap acceptance and flared approaches based on empirical evidence of their effects.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

The identified limitations for this chapter are shown in Exhibit 20-4, along with the potential for improved treatment by alternative tools.

Exhibit 20-4
Limitations of the HCM TWSC Intersection Motorized Vehicle Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Effects of upstream intersections	Simulation tools can include an unsignalized intersection explicitly within a signalized arterial or network.
YIELD-controlled intersection operations	Treated explicitly by some tools. Can be approximated by varying the gap acceptance parameters.
Non-steady-state conditions for demand and capacity	Most alternative tools provide for multiperiod variation of demand and, in some cases, capacity.
Atypical intersection configurations, such as more than four legs or STOP control on all but one leg	Some alternative tools can be customized to model the unique configuration of these types of intersections.

The most common application of alternative tools for TWSC analysis involves an unsignalized intersection located between coordinated signalized intersections.

Most analyses for isolated unsignalized intersections are intended to determine whether TWSC is a viable control alternative. Analyses of this type are handled adequately by the procedures described in this chapter. The most common application of alternative tools for TWSC analysis involves an unsignalized intersection located between coordinated signalized intersections. Most intersections between coordinated signalized intersections operate under TWSC. These intersections tend to be ignored in the analysis of the system because their effect on the system operation is minimal. Occasionally, it is necessary to examine a TWSC intersection as a part of the arterial system. Although the procedures in this chapter provide a method for approximating the operation of a TWSC intersection with an upstream signal, the operation of such an intersection is arguably best handled by including it in a complete simulation of the full arterial system. For example, queue backup from a downstream signal that blocks entry from the cross street for a portion of the cycle is not treated explicitly by the procedures contained in this chapter.

Another potential application for alternative tools is modeling intersections with more than four legs or with control configurations other than the typical priority control, such as STOP control on all but one leg. The operation of these types of intersections has not been adequately researched, and no analytical method has been developed to model their operation. It may be possible to use an alternative tool to model these configurations provided the priorities between movements can be customized to match field operations.

Development of HCM-Compatible Performance Measures Using Alternative Tools

Control delay, the performance measure that determines LOS for TWSC intersections, is defined as that portion of the delay caused by a control device—in this case, a STOP sign. Most simulation tools do not produce explicit estimates of control delay.

The best way to determine control delay at a STOP sign from simulation is to perform simulation runs with and without the control device in place. The segment delays reported with no control represent the delays due to geometrics and interaction between vehicles. The additional delay reported in the run with the control device in place is, by definition, the control delay.

Chapter 7, Interpreting HCM and Alternative Tool Results, discusses performance measures from various tools in more detail, and Chapter 36, Concepts: Supplemental, provides recommendations on how individual vehicle trajectories should be interpreted to produce specific performance measures. Of particular interest to TWSC operation is the definition of a queued state and the development of queue delay from that definition. For alternative tools that conform to the queue delay definitions and computations presented in this manual, queue delay will provide the best estimate of control delay for TWSC intersections. Delay and LOS should not be estimated by using alternative tools that do not conform to these definitions and computations.

Delay and LOS should be estimated only by using alternative tools that conform to the definitions and computations of queue delay presented in this manual.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Deterministic tools and simulation tools both model TWSC operations as a gap acceptance process that follows the rules of the road to determine the right-of-way hierarchy. To this extent, both types of tools use the same conceptual framework. Deterministic tools such as the HCM base their estimates of capacity and delay on expected values computed from analytical formulations that have been mathematically derived. Simulation tools, in contrast, take a more microscopic view, treating each vehicle as an independent object that is subject to the rules of the road as well as interaction with other vehicles. Differences in the treatment of randomness also exist, as explained in the guidance provided in Chapter 19, Signalized Intersections.

When the opposing movement volumes are very high, there is minimal opportunity for the STOP-controlled movements to accept gaps, and these movements often have little or no capacity. Simulation tends to produce slightly higher capacities under these conditions because of a tool-specific overriding logic that limits the amount of time any driver is willing to wait for a gap.

In general, the simulation results for a specific TWSC intersection problem should be close to the results obtained from the procedures in this chapter. Some differences may, however, be expected among the analysis tools.

Adjustment of Simulation Parameters to the HCM Parameters

Critical headways and follow-up headways are common to both deterministic and simulation models. It is therefore desirable that similar values be used for these parameters.

Sample Calculations Illustrating Alternative Tool Applications

An example of the most common application for TWSC simulation, unsignalized intersections within a signalized arterial system, is presented in Chapter 29, Urban Street Facilities: Supplemental. An additional example involving blockage of a cross-street approach with STOP control by a queue from a nearby diamond interchange is presented in Chapter 34, Interchange Ramp Terminals: Supplemental.

REQUIRED INPUT DATA AND SOURCES

Exhibit 20-5 lists the information necessary to apply the motorized vehicle methodology and suggests potential sources for obtaining these data. It also suggests default values for use when intersection-specific information is not available.

Exhibit 20-5
Required Input Data, Potential Data Sources, and Default Values for TWSC Motorized Vehicle Analysis

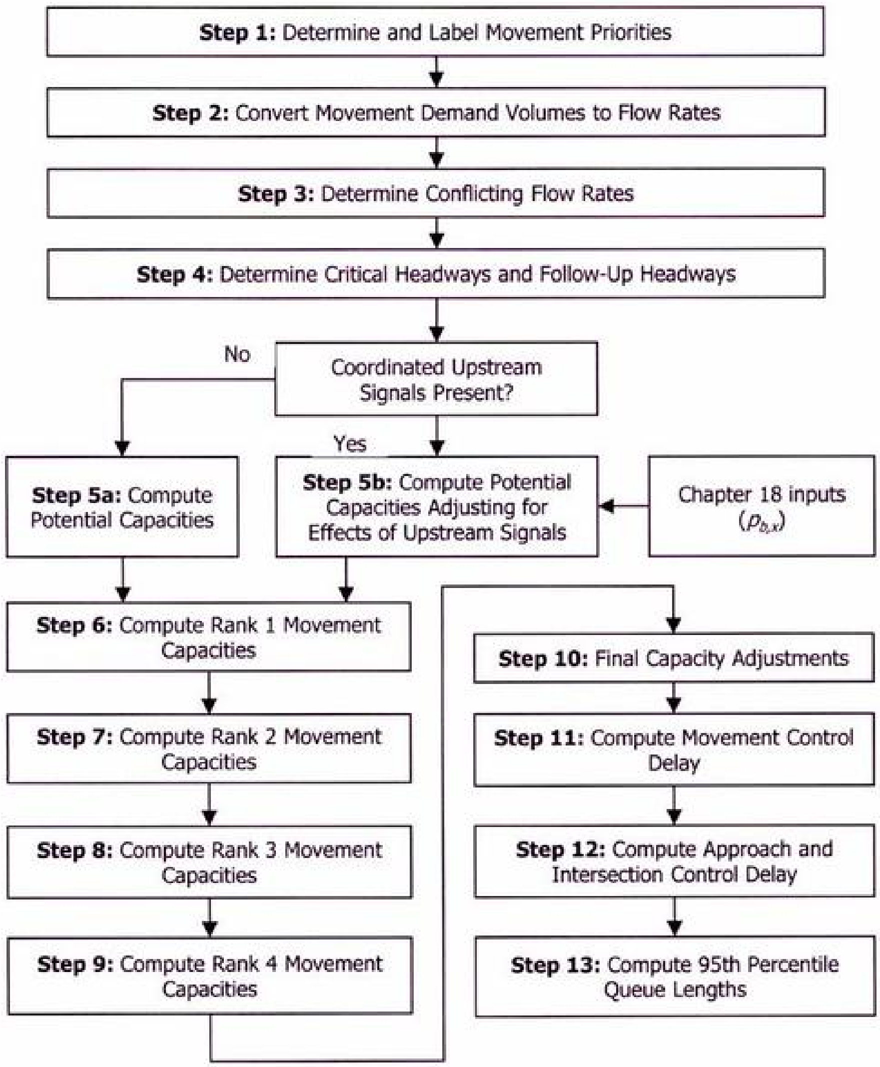
Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Number and configuration of lanes of each approach	Design plans, road inventory	Must be provided
Approach grades	Design plans, road inventory	0%
Special geometric factors such as • Unique channelization aspects • Existence of a two-way left-turn lane or raised or striped median storage (or both) • Existence of flared approaches on the minor street • Existence of upstream signals	Design plans, road inventory	Must be provided
	Demand Data	
Hourly turning-movement demand volume (veh/h) AND peak hour factor OR Hourly turning-movement demand flow rate (veh/h)	Field data, modeling	Must be provided
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor (decimal)	Field data	0.92
Heavy-vehicle percentage (%)	Field data	3%
Saturation flow rate for major-street through movement (for analysis of shared or short major-street, left-turn lanes)	Field data	1,800 veh/h
Saturation flow rate for major-street, right-turn movement (for analysis of shared or short major-street, left-turn lanes)	Field data	1,500 veh/h

A comprehensive presentation of potential default values for interrupted flow facilities is provided elsewhere (*5*), with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of peak hour factor and percentage of heavy vehicles. Recommendations are based on geographic region, population, and time of day. All general default values for interrupted-flow facilities may be applied to the analysis of TWSC intersections in the absence of field data or projected conditions.

COMPUTATIONAL STEPS

The TWSC intersection methodology for the motorized vehicle mode is applied through a series of steps that require input data related to movement flow information and geometric conditions, prioritization of movements, computation of potential capacities, incorporation of adjustments to compute movement capacities, and estimation of control delays and queue lengths. These steps are illustrated in Exhibit 20-6.

Exhibit 20-6 TWSC Intersection Methodology



Step 1: Determine and Label Movement Priorities

The priority for each movement at a TWSC intersection must be identified to designate the appropriate rank of each movement for future steps in the analysis process. This step's process also identifies the sequence in which the analyst will complete the capacity computations. Because the methodology is based on prioritized use of gaps by vehicles at a TWSC intersection, the subsequent computations must be made in a precise order. The computational sequence is the same as the priority of gap use, and movements are considered in the following order:

1. Left turns from the major street,
2. Right turns from the minor street,
3. U-turns from the major street,
4. Through movements from the minor street, and
5. Left turns from the minor street.

Step 2: Convert Movement Demand Volumes to Flow Rates

For analysis of existing conditions when the peak 15-min period can be measured in the field, the volumes for the peak 15-min period are converted to a peak 15-min demand flow rate by multiplying the peak 15-min volumes by four.

For analysis of projected conditions or when 15-min data are not available, hourly demand volumes for each movement are converted to peak 15-min demand flow rates in vehicles per hour, as shown in Equation 20-1, through use of the peak hour factor for the intersection.

Equation 20-1

$$v_i = \frac{V_i}{PHF}$$

where

v_i = demand flow rate for movement i (veh/h),

V_i = demand volume for movement i (veh/h), and

PHF = peak hour factor for the intersection.

If PHF is used, a single intersectionwide PHF should be used rather than movement-specific or approach-specific PHFs. If individual approaches or movements peak at different times, a series of 15-min analysis periods that encompasses the peaking should be considered.

If peak hour factors are used, a single peak hour factor for the entire intersection is generally preferred to decrease the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, but in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. If individual approaches or movements are known to have substantially different peaking characteristics or peak during different 15-min periods within the hour, a series of 15-min analysis periods that encompasses the peaking should be considered instead of a single analysis period using a single peak hour factor for the intersection.

The use of a peak 15-min traffic count multiplied by four is preferred for existing conditions when traffic counts are available. The use of a 1-h demand volume divided by a peak hour factor is preferred with projected volumes or with projected volumes that have been added to current volumes.

Step 3: Determine Conflicting Flow Rates

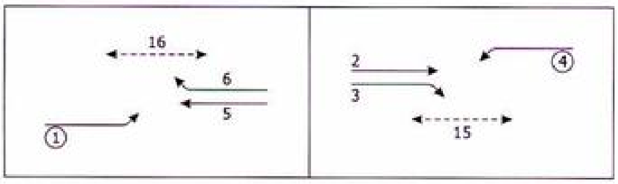
Each movement at a TWSC intersection faces a different set of conflicts that is directly related to the nature of the subject movement. The following subsections describe the set of conflicts facing each minor movement (Rank 2 through Rank 4) at a TWSC intersection. The exhibits and equations illustrate the computation of the parameter $v_{c,x}$, the conflicting flow rate for movement x—that is, the total flow rate (in vehicles per hour) that conflicts with movement x.

Pedestrians may also conflict with vehicular movements. Pedestrian flow rates (defined as v_x, with x noting the leg of the intersection being crossed) should be included as part of the conflicting flow rates. Pedestrian flows are included because they define the beginning or ending of a gap that may be used by a minor-street movement. Although this method recognizes some peculiarities associated with pedestrian movements, it takes a uniform approach to vehicular and pedestrian movements.

Major-Street Left-Turn Movements: Rank 2, Movements 1 and 4

Exhibit 20-7 illustrates the conflicting movements and Equation 20-2 and Equation 20-3 compute the conflicting flow rates encountered by major-street left-turning drivers. The left-turn movement from the major street conflicts with the total opposing through and right-turn flow, because the left-turning vehicles must cross the opposing through movement and be in conflict with the right-turning vehicles. The method does not differentiate between crossing and merging conflicts. Left-turning vehicles from the major street and the opposing right-turning vehicles from the major street are considered to merge, regardless of the number of lanes provided in the exit roadway.

Exhibit 20-7
Illustration of Conflicting Movements for Major-Street Left-Turn Movements



$$v_{c,1} = v_5 + v_6 + v_{16}$$ **Equation 20-2**

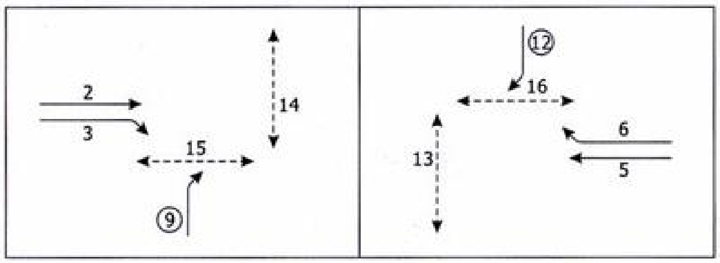
$$v_{c,4} = v_2 + v_3 + v_{15}$$ **Equation 20-3**

If the major-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, then the v_6 and v_3 terms in Equation 20-2 and Equation 20-3, respectively, may be assumed to be zero.

Minor-Street Right-Turn Movements: Rank 2, Movements 9 and 12

Exhibit 20-8 illustrates the conflicting movements encountered by minor-street right-turning drivers. The right-turn movement from the minor street is assumed to be in conflict with only a portion of the major-street through movement when more than one major-street lane is present. Also, one-half of each right-turn movement from the major street is considered to conflict with the minor-street right-turn movement, as some of these turns tend to inhibit the subject movement. Because right-turning vehicles from the minor street commonly merge into gaps in the right-hand lane of the stream into which they turn, they typically do not require a gap across all lanes of the conflicting stream (this situation may not be true for some trucks and vans with long wheelbases that encroach on more than one lane in making their turns). Furthermore, a gap in the overall major-street traffic could be used simultaneously by another vehicle, such as a major-street left-turning vehicle. Exhibit 20-8 does not include vehicles making major-street U-turns as conflicting vehicles. Although these conflicts may be observed in practice, they are not assumed to be conflicts in this methodology.

Exhibit 20-8
Illustration of Conflicting Movements for Minor-Street Right-Turn Movements



Equation 20-4 through Equation 20-9 compute the conflicting flow rates for minor-street right-turn movements entering a major street. If the major-street right turn has its own lane, the corresponding v_3 or v_6 term in these equations may be assumed to be zero. Users may supply different lane distributions for the v_2 and v_5 terms in the equations for four- and six-lane major streets when supported by field data.

Equation 20-4 and Equation 20-5 compute the conflicting flow rates for minor-street right-turn movements entering two-lane major streets:

Equation 20-4

$$v_{c,9} = v_2 + 0.5v_3 + v_{14} + v_{15}$$

Equation 20-5

$$v_{c,12} = v_5 + 0.5v_6 + v_{13} + v_{16}$$

Equation 20-6 and Equation 20-7 are used for four-lane major streets:

Equation 20-6

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15}$$

Equation 20-7

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{13} + v_{16}$$

Equation 20-8 and Equation 20-9 are used for six-lane major streets:

Equation 20-8

$$v_{c,9} = 0.5v_2 + 0.5v_3 + v_{14} + v_{15}$$

Equation 20-9

$$v_{c,12} = 0.5v_5 + 0.5v_6 + v_{13} + v_{16}$$

Major-Street U-Turn Movements: Rank 2, Movements 1U and 4U

Exhibit 20-9 illustrates the conflicting movements encountered by major-street U-turning drivers. The U-turn movement from the major street conflicts with the total opposing through and right-turn flow, similar to the major-street left-turn movement. Research has found that the presence of minor-street right-turning vehicles significantly affects the capacity of major-street U-turns (*6*). The methodology accounts for this effect in the impedance calculation rather than in the calculation of conflicting flow. If a different priority order is desired (e.g., minor-street right turns yield to major-street U-turns), the analyst should adjust the computation procedure accordingly to replicate observed conditions.

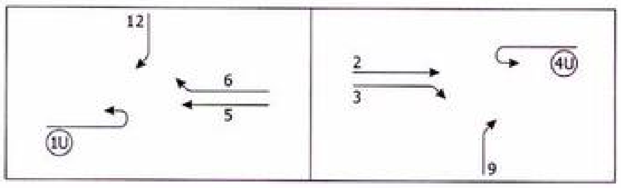


Exhibit 20-9
Illustration of Conflicting Movements for Major-Street U-Turn Movements

Equation 20-10 through Equation 20-13 compute the conflicting flow rates for major-street U-turns. No field data are available for U-turns on major streets with fewer than four lanes. If a major-street right turn has its own lane, the corresponding v_3 or v_6 term in these equations should be assumed to be zero.

Equation 20-10 and Equation 20-11 compute the conflicting flow rates for major-street U-turns when the major street has four lanes:

$$v_{c,1U} = v_5 + v_6$$

Equation 20-10

$$v_{c,4U} = v_2 + v_3$$

Equation 20-11

Equation 20-12 and Equation 20-13 compute the conflicting flow rates for major-street U-turns on six-lane major streets:

$$v_{c,1U} = 0.73v_5 + 0.73v_6$$

Equation 20-12

$$v_{c,4U} = 0.73v_2 + 0.73v_3$$

Equation 20-13

Minor-Street Pedestrian Movements: Rank 2, Movements 13 and 14

Minor-street pedestrian movements (those pedestrians crossing the major street) are in direct conflict with all vehicular movements on the major street except the right-turn and left-turn movements on the major street approaching from the far side of the intersection. The volume of minor-street pedestrians is an input parameter in the computation of the conflicting flow rates for all Rank 3 and Rank 4 movements.

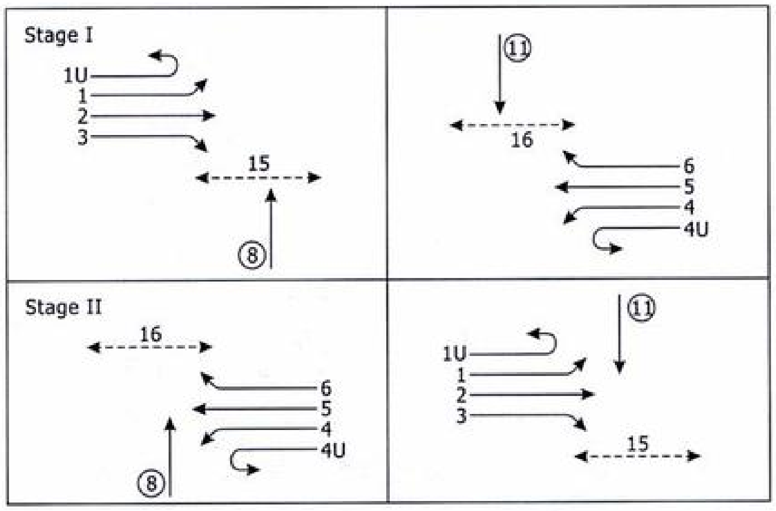
Minor-Street Through Movements: Rank 3, Movements 8 and 11

Minor-street through movements have a direct crossing or merging conflict with all movements on the major street except the right turn into the subject approach. Similar to the minor-street right-turn movement, one-half of each right-turn movement from the major street is considered to conflict with the minor-street through movement. In addition, field research (*1*) has shown that the effect of left-turning vehicles is approximately twice their actual number.

Drivers executing minor-street through movements may complete their maneuver in one or two stages. One-stage gap acceptance assumes no median refuge area is available for minor-street drivers to store in and that the minor-street drivers will evaluate gaps in both major-street directions simultaneously. Conversely, the two-stage gap acceptance scenario assumes a median refuge area is available for minor-street drivers. During Stage I, minor-street drivers evaluate major-street gaps in the nearside traffic stream (conflicting traffic from the left); during Stage II, minor-street drivers evaluate major-street gaps in the farside traffic stream (conflicting traffic from the right). For one-stage crossings, the conflicting flows for Stage I and Stage II are combined; for two-stage crossings, the conflicting flows are considered separately.

Exhibit 20-10 illustrates the conflicting movements encountered by minor-street through-movement drivers.

Exhibit 20-10
Illustration of Conflicting Movements for Minor-Street Through Movements



Equation 20-14 and Equation 20-15 compute the conflicting flows encountered by minor-street through-movement drivers during Stage I. If there is a right-turn lane on the major street, the corresponding v_3 or v_6 term in these equations may be assumed to be zero.

Equation 20-14

$$v_{c,\mathrm{I},8} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$

Equation 20-15

$$v_{c,\mathrm{I},11} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$

Equation 20-16 and Equation 20-17 compute the conflicting flows encountered by minor-street through-movement drivers during Stage II. If the major-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, the corresponding v_3 or v_6 term in these equations may be assumed to be zero.

Equation 20-16

$$v_{c,\mathrm{II},8} = 2(v_4 + v_{4U}) + v_5 + v_6 + v_{16}$$

Equation 20-17

$$v_{c,\mathrm{II},11} = 2(v_1 + v_{1U}) + v_2 + v_3 + v_{15}$$

Minor-Street Left-Turn Movements: Rank 4, Movements 7 and 10

The left-turn movement from the minor street is the most difficult maneuver to execute at a TWSC intersection, and it faces the most complex set of conflicting movements, which include all major-street movements in addition to the opposing right-turn and through movements on the minor street. Only one-half the opposing right-turn and through-movement flow rate is included as conflicting flow rate because both movements are STOP-controlled, which diminishes their effect on left turns. The additional capacity impedance effects of the opposing right-turn and through-movement flow rates are taken into account elsewhere in the procedure.

Similar to minor-street through movements, minor-street left-turn movements may be completed in one or two stages. Exhibit 20-11 illustrates the conflicting movements encountered by minor-street left-turning drivers.

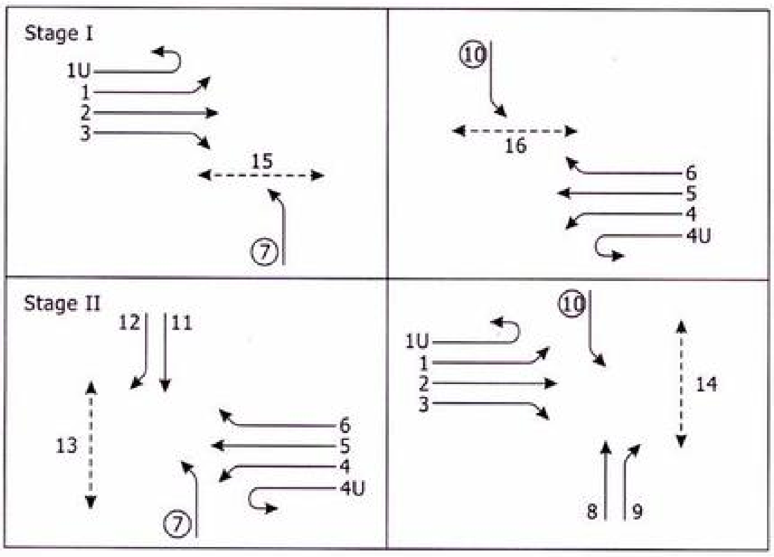


Exhibit 20-11
Illustration of Conflicting Movements for Minor-Street Left-Turn Movements

Equation 20-18 through Equation 20-23 compute the conflicting flow rates for minor-street left-turn movements entering a major street during Stage I. If a right-turn lane exists on the major street, the corresponding v_3 or v_6 term in these equations may be assumed to be zero.

During Stage I, Equation 20-18 and Equation 20-19 compute the conflicting flow rates for minor-street left-turn movements entering two-lane major streets:

$$v_{c,\mathrm{I},7} = 2v_1 + v_2 + 0.5v_3 + v_{15}$$ **Equation 20-18**

$$v_{c,\mathrm{I},10} = 2v_4 + v_5 + 0.5v_6 + v_{16}$$ **Equation 20-19**

Equation 20-20 and Equation 20-21 are used for four-lane major streets:

$$v_{c,\mathrm{I},7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$ **Equation 20-20**

$$v_{c,\mathrm{I},10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$ **Equation 20-21**

Equation 20-22 and Equation 20-23 are used for six-lane major streets:

$$v_{c,\mathrm{I},7} = 2(v_1 + v_{1U}) + v_2 + 0.5v_3 + v_{15}$$ **Equation 20-22**

$$v_{c,\mathrm{I},10} = 2(v_4 + v_{4U}) + v_5 + 0.5v_6 + v_{16}$$ **Equation 20-23**

Similarly, Equation 20-24 though Equation 20-29 compute the conflicting flow rates for minor-street left-turn movements entering a major street during Stage II. If the minor-street right turn is separated by a triangular island and has to comply with a YIELD or STOP sign, the corresponding v_9 or v_{12} term in these equations may be assumed to be zero.

During Stage II, Equation 20-24 and Equation 20-25 compute the conflicting flow rates for minor-street left-turn movements entering two-lane major streets:

$$v_{c,\mathrm{II},7} = 2v_4 + v_5 + 0.5v_6 + 0.5v_{12} + 0.5v_{11} + v_{13}$$ **Equation 20-24**

$$v_{c,\mathrm{II},10} = 2v_1 + v_2 + 0.5v_3 + 0.5v_9 + 0.5v_8 + v_{14}$$ **Equation 20-25**

Equation 20-26 and Equation 20-27 are used for four-lane major streets:

$$v_{c,\mathrm{II},7} = 2(v_4 + v_{4U}) + 0.5v_5 + 0.5v_{11} + v_{13}$$ **Equation 20-26**

$$v_{c,\mathrm{II},10} = 2(v_1 + v_{1U}) + 0.5v_2 + 0.5v_8 + v_{14}$$ **Equation 20-27**

Equation 20-28 and Equation 20-29 are used for six-lane major streets:

Equation 20-28

$$v_{c,\mathrm{II},7} = 2(v_4 + v_{4U}) + 0.4v_5 + 0.5v_{11} + v_{13}$$

Equation 20-29

$$v_{c,\mathrm{II},10} = 2(v_1 + v_{1U}) + 0.4v_2 + 0.5v_8 + v_{14}$$

Step 4: Determine Critical Headways and Follow-Up Headways

The critical headways $t_{c,x}$ and follow-up headways $t_{f,x}$ must be determined for the major-street left turns ($v_{c,1}$ and $v_{c,4}$), the minor-street right turns ($v_{c,9}$ and $v_{c,12}$), the major-street U-turns ($v_{c,1U}$ and $v_{c,4U}$), the minor-street through movements ($v_{c,8}$ and $v_{c,11}$), and the minor-street left turns ($v_{c,7}$ and $v_{c,10}$) as they occur at a TWSC intersection.

To compute the critical headways for each movement, the analyst begins with the base critical headway given in Exhibit 20-12 and makes movement-specific adjustments relating to the percentage of heavy vehicles, the grade encountered, and a three-leg versus four-leg intersection as shown in Equation 20-30.

Equation 20-30

$$t_{c,x} = t_{c,\mathrm{base}} + t_{c,HV}P_{HV} + t_{c,G}G - t_{3,LT}$$

where

$t_{c,x}$ = critical headway for movement x (s),

$t_{c,\mathrm{base}}$ = base critical headway from Exhibit 20-12 (s),

$t_{c,HV}$ = adjustment factor for heavy vehicles (1.0 for major streets with one lane in each direction; 2.0 for major streets with two or three lanes in each direction) (s),

P_{HV} = proportion of heavy vehicles for movement (expressed as a decimal; e.g., $P_{HV} = 0.02$ for 2% heavy vehicles),

$t_{c,G}$ = adjustment factor for grade for given movement (0.1 for Movements 9 and 12; 0.2 for Movements 7, 8, 10, and 11) (s),

G = percentage grade (expressed as an integer; e.g., $G = -2$ for a 2% downhill grade), and

$t_{3,LT}$ = 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 20-12
Base Critical Headways for TWSC Intersections

Vehicle Movement	Base Critical Headway, $t_{c,\mathrm{base}}$ (s) Two Lanes	Four Lanes	Six Lanes
Left turn from major street	4.1	4.1	5.3
U-turn from major street	NA	6.4 (wide)[a] 6.9 (narrow)[a]	5.6
Right turn from minor street	6.2	6.9	7.1
Through traffic on minor street	1 stage: 6.5 2 stage, Stage I: 5.5 2 stage, Stage II: 5.5	1 stage: 6.5 2 stage, Stage I: 5.5 2 stage, Stage II: 5.5	1 stage: 6.5[b] 2 stage, Stage I: 5.5[b] 2 stage, Stage II: 5.5[b]
Left turn from minor street	1 stage: 7.1 2 stage, Stage I: 6.1 2 stage, Stage II: 6.1	1 stage: 7.5 2 stage, Stage I: 6.5 2 stage, Stage II: 6.5	1 stage: 6.4 2 stage, Stage I: 7.3 2 stage, Stage II: 6.7

Notes: NA = not available.
[a] Narrow U-turns have a median nose width <21 ft; wide U-turns have a median nose width ≥21 ft.
[b] Use caution; values estimated.

The critical headway data for four- and six-lane sites account for the actual lane distribution of traffic flows measured at each site. For six-lane sites, minor-street left turns were commonly observed beginning their movement while apparently conflicting vehicles in the farside major-street through stream passed. The values for critical headway for minor-street through movements at six-lane streets are estimated, as the movement is not frequently observed in the field.

Similar to the computation of critical headways, the analyst begins the computation of follow-up headways with the base follow-up headways given in Exhibit 20-13. The analyst then makes movement-specific adjustments to the base follow-up headways with information gathered on heavy vehicles and the geometrics of the major street per the adjustment factors given in Equation 20-31.

Equation 20-31

$$t_{f,x} = t_{f,\text{base}} + t_{f,HV} P_{HV}$$

where

$t_{f,x}$ = follow-up headway for movement x (s),

$t_{f,\text{base}}$ = base follow-up headway from Exhibit 20-13 (s),

$t_{f,HV}$ = adjustment factor for heavy vehicles (0.9 for major streets with one lane in each direction; 1.0 for major streets with two or three lanes in each direction), and

P_{HV} = proportion of heavy vehicles for movement (expressed as a decimal; e.g., $P_{HV} = 0.02$ for 2% heavy vehicles).

Exhibit 20-13
Base Follow-Up Headways for TWSC Intersections

	Base Follow-Up Headway, $t_{f,\text{base}}$ (s)		
Vehicle Movement	**Two Lanes**	**Four Lanes**	**Six Lanes**
Left turn from major street	2.2	2.2	3.1
U-turn from major street	NA	2.5 (wide)[a] 3.1 (narrow)[a]	2.3
Right turn from minor street	3.3	3.3	3.9
Through traffic on minor street	4.0	4.0	4.0
Left turn from minor street	3.5	3.5	3.8

Notes: NA = not available.
[a] Narrow U-turns have a median nose width <21 ft; wide U-turns have a median nose width ≥21 ft.

Values from Exhibit 20-12 and Exhibit 20-13 are based on studies throughout the United States and are representative of a broad range of conditions. If smaller values for t_c and t_f are observed, capacity will be increased. If larger values for t_c and t_f are used, capacity will be decreased.

Step 5: Compute Potential Capacities

Step 5a: Potential Capacity Without Upstream Signal Effects

The potential capacity $c_{p,x}$ of a movement is computed according to the gap acceptance model provided in Equation 20-32 (7).

Equation 20-32

$$c_{p,x} = v_{c,x} \frac{e^{-v_{c,x} t_{c,x}/3{,}600}}{1 - e^{-v_{c,x} t_{f,x}/3{,}600}}$$

where

$c_{p,x}$ = potential capacity of movement x (veh/h),

$v_{c,x}$ = conflicting flow rate for movement x (veh/h),

$t_{c,x}$ = critical headway for minor movement x (s), and

$t_{f,x}$ = follow-up headway for minor movement x (s).

For two-stage Rank 3 or Rank 4 movements, the potential capacity is computed three times: $c_{p,x}$ assuming one-stage operation, $c_{p,I,x}$ 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 with Upstream Signal Effects

To evaluate the impact of coordinated upstream signals, the urban street segments methodology (Chapter 17) is used to estimate the proportion of time that each Rank 2 or lower movement will be effectively blocked by a platoon. The proportion of time blocked is denoted by $p_{b,x}$, where x is the movement using the movement conventions provided in Exhibit 20-1.

With these values, the proportion of the analysis period that is blocked for each minor movement can be computed by using Exhibit 20-14.

Exhibit 20-14
Proportion of Analysis Period Blocked for Each Movement

	Proportion Blocked for Movement, $p_{b,x}$		
		Two-Stage Movements	
Movement(s) x	**One-Stage Movements**	**Stage I**	**Stage II**
1, 1U	$p_{b,1}$	NA	NA
4, 4U	$p_{b,4}$	NA	NA
7	$p_{b,7}$	$p_{b,4}$	$p_{b,1}$
8	$p_{b,8}$	$p_{b,4}$	$p_{b,1}$
9	$p_{b,9}$	NA	NA
10	$p_{b,10}$	$p_{b,1}$	$p_{b,4}$
11	$p_{b,11}$	$p_{b,1}$	$p_{b,4}$
12	$p_{b,12}$	NA	NA

Note: NA = not applicable.

The flow for the unblocked period (no platoons) is determined in this step. This flow becomes the conflicting flow for the subject movement and is used to compute the capacity for this movement. The minimum platooned flow rate $v_{c,min}$ is approximately 1,000N, where N is the number of through lanes per direction on the major street (*8*).

The conflicting flow for movement x during the unblocked period is given by Equation 20-33.

Equation 20-33

$$v_{c,u,x} = \begin{cases} \dfrac{v_{c,x} - 1.5 v_{c,min} p_{b,x}}{1 - p_{b,x}} & \text{if } v_{c,x} > 1.5 v_{c,min} p_{b,x} \\ 0 & \text{otherwise} \end{cases}$$

where

$v_{c,u,x}$ = conflicting flow for movement x during the unblocked period (veh/h);

$v_{c,x}$ = total conflicting flow for movement x as determined from Step 3 (veh/h);

$v_{c,min}$ = minimum platooned flow rate (veh/h), assumed to be 1,000N, where N is the number of through lanes per direction on the major street; and

$p_{b,x}$ = proportion of time the subject movement x is blocked by the major-street platoon, which is determined from Exhibit 20-14.

The potential capacity of the subject movement x, accounting for the effect of platooning, is given by Equation 20-34 and Equation 20-35.

$$c_{p,x} = (1 - p_{b,x})c_{r,x}$$

Equation 20-34

$$c_{r,x} = v_{c,u,x} \frac{e^{-v_{c,u,x}t_{c,x}/3{,}600}}{1 - e^{-v_{c,u,x}t_{f,x}/3{,}600}}$$

Equation 20-35

where

$c_{p,x}$ = potential capacity of movement x (veh/h),

$p_{b,x}$ = proportion of time that movement x is blocked by a platoon, and

$c_{r,x}$ = capacity of movement x assuming random flow during the unblocked period.

These equations use the same critical headway and follow-up headway inputs as a normal calculation, but they use only the conflicting flow during the unblocked period.

Steps 6–9: Compute Movement Capacities

For clarity, these steps assume pedestrian impedance effects can be neglected, and in many cases this assumption is reasonable. However, pedestrians can be accounted for in the analysis of the motorized vehicle mode by replacing these steps with those provided in Section 4, Extension to the Motorized Vehicle Methodology, which incorporate the effects of pedestrian impedance.

Step 6: Compute Rank 1 Movement Capacities

Rank 1 major-street movements are assumed to be unimpeded by any movements of lower rank. This rank also implies that major-street movements of Rank 1 are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays do occasionally occur, and they are accounted for by using adjustments provided later in this procedure.

Step 7: Compute Rank 2 Movement Capacities

Movements of Rank 2 (left turns and U-turns from the major street and right turns from the minor street) must yield to conflicting major-street through and right-turning vehicular movements of Rank 1. Minor-street right turns are assumed to yield to major-street U-turns, although sometimes the reverse occurs.

Step 7a: Movement Capacity for Major-Street Left-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 major-street left-turn movements (1 and 4) is equal to its potential capacity $c_{p,j}$, as shown in Equation 20-36.

$$c_{m,j} = c_{p,j}$$

Equation 20-36

Step 7b: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 minor-street right-turn movements (9 and 12) is equal to its potential capacity $c_{p,j}$, as shown in Equation 20-37.

$$c_{m,j} = c_{p,j}$$

Equation 20-37

Step 7c: Movement Capacity for Major-Street U-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 major-street U-turn movements (1U and 4U) is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. Field observations are mixed in terms of the degree to which major-street U-turn movements yield to minor-street right-turn movements and vice versa (5). It is assumed that the presence of minor-street right-turning vehicles will impede U-turning vehicles from accepting gaps in the major-street traffic stream; therefore, the capacity of the U-turn movement is affected by the probability that the minor-street right-turning traffic will operate in a queue-free state. The capacity adjustment factors are denoted by f_{1U} and f_{4U} for the major-street U-turn movements 1U and 4U, respectively, and are given by Equation 20-38 and Equation 20-39, respectively.

Equation 20-38

$$f_{1U} = p_{0,12} = 1 - \frac{v_{12}}{c_{m,12}}$$

Equation 20-39

$$f_{4U} = p_{0,9} = 1 - \frac{v_9}{c_{m,9}}$$

where

f_{1U}, f_{4U} = capacity adjustment factor for Rank 2 major-street U-turn movements 1 and 4, respectively;

$p_{0,j}$ = probability that conflicting Rank 2 minor-street right-turn movement j will operate in a queue-free state;

v_j = flow rate of movement j;

$c_{m,j}$ = capacity of movement j; and

j = 9 and 12 (minor-street right-turn movements of Rank 2).

The movement capacity for major-street U-turn movements is then computed with Equation 20-40.

Equation 20-40

$$c_{m,jU} = c_{p,jU} \times f_{jU}$$

In almost all cases, major-street left-turning vehicles share a lane with U-turning vehicles. If Rank 2 major-street U-turn movements are present to a significant degree, then Equation 20-41 should be used to compute the shared-lane capacity.

where

$c_{m,jU}$ = movement capacity for Movements 1U and 4U,

$c_{p,jU}$ = potential capacity for Movements 1U and 4U (from Step 5), and

f_{jU} = capacity adjustment factor for Movements 1U and 4U.

Because left-turn and U-turn movements are typically made from the same lane, their shared-lane capacity is computed with Equation 20-41.

Equation 20-41

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}}$$

where

c_{SH} = capacity of the shared lane (veh/h),

v_y = flow rate of the y movement in the subject shared lane (veh/h), and

$c_{m,y}$ = movement capacity of the y movement in the subject shared lane (veh/h).

Step 7d: Effect of Major-Street Shared Through and Left-Turn Lane

The probability that the major-street left-turning traffic will operate in a queue-free state is expressed by Equation 20-42.

$$p_{0,j} = 1 - \frac{v_j}{c_{m,j}}$$

Equation 20-42

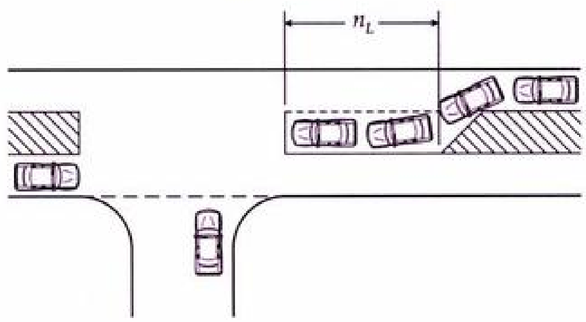
where j = 1 and 4 (major-street left-turn and U-turn movements of Rank 2, using shared volume and capacity as appropriate).

Use Equation 20-42 to compute the probability of a queue-free state for Rank 2 movements.

If major-street through and left-turn movements are shared, use Equation 20-43.

If, however, a shared left-turn lane or a short left-turn pocket is present on a major-street approach (as in Exhibit 20-15), the analyst accounts for this occurrence by computing the probability that there will be no queue in the major-street shared lane, $p^*_{0,j}$, according to Equation 20-43. This probability is then used by the analyst in lieu of $p_{0,j}$ from Equation 20-42.

Exhibit 20-15
Short Left-Turn Pocket on Major-Street Approach



The methodology implicitly assumes an exclusive lane is provided to all left-turning traffic from the major street. If a left-turn lane is not provided or the left-turn pocket is not long enough to accommodate all queuing left-turn and U-turn vehicles, major-street through (and possibly right-turning) traffic could be delayed by left-turning vehicles waiting for an acceptable gap in opposing major-street through traffic. To account for this occurrence, the factors $p^*_{0,1}$ and $p^*_{0,4}$ may be computed according to Equation 20-43 and Equation 20-44 as an indication of the probability there will be no queue in the respective major-street shared or short lanes (9).

$$p^*_{0,j} = 1 - \left(1 - p_{0,j}\right)\left[\sqrt[(n_L+1)]{1 + \frac{x_{i,1+2}^{(n_L+1)}}{1 - x_{i,1+2}}}\right]$$

Equation 20-43

$$x_{i,1+2} = \frac{v_{i1}}{s_{i1}} + \frac{v_{i2}}{s_{i2}}$$

Equation 20-44

where

$p_{0,j}$ = probability of queue-free state for movement j assuming an exclusive left-turn lane on the major street (per Equation 20-42);

$p^*_{0,j}$ = probability of queue-free state for movement j assuming a shared left-turn lane on the major street;

j = 1 and 4 (major-street left-turning vehicular movements);

$i1$ = 2 and 5 (major-street through vehicular movements);

$i2$ = 3 and 6 (major-street right-turning vehicular movements);

When j = 1, $i1$ = 2 and $i2$ = 3; when j = 4, $i1$ = 5 and $i2$ = 6.

$x_{i,1+2}$ = combined degree of saturation for the major-street through and right-turn movements;

s_{i1} = saturation flow rate for the major-street through movements (default assumed to be 1,800 veh/h; however, this parameter can be measured in the field);

s_{i2} = saturation flow rate for the major-street right-turn movements (default assumed to be 1,500 veh/h; however, this parameter can be measured in the field);

v_{i1} = major-street through-movement flow rate (veh/h);

v_{i2} = major-street right-turn flow rate (veh/h) (0 if an exclusive right-turn lane is provided); and

n_L = number of vehicles that can be stored in the left-turn pocket (see Exhibit 20-15).

For the special situation of shared lanes ($n_L = 0$), Equation 20-43 becomes Equation 20-45 as follows:

Equation 20-45

$$p_{0,j}^* = 1 - \frac{1 - p_{0,j}}{1 - x_{i,1+2}}$$

where all terms are as previously defined.

By using $p_{0,1}^*$ and $p_{0,4}^*$ in lieu of $p_{0,1}$ and $p_{0,4}$ (as computed by Equation 20-42), the potential for queues on a major street with shared or short left-turn lanes may be taken into account.

Step 8: Compute Rank 3 Movement Capacities

Rank 3 minor-street traffic movements (minor-street through movements at four-leg intersections and minor-street left turns at three-leg intersections) must yield to conflicting Rank 1 and Rank 2 movements. Not all gaps of acceptable length that pass through the intersection will normally be available for use by Rank 3 movements, because some of these gaps are likely to be used by Rank 2 movements.

If the Rank 3 movement is a two-stage movement, the movement capacity for the one-stage movement is computed as an input to the two-stage calculation.

Step 8a: Rank 3 Capacity for One-Stage Movements

For Rank 3 movements, the magnitude of vehicle impedance depends on the probability that major-street left-turning vehicles will be waiting for an acceptable gap at the same time as vehicles of Rank 3. A higher probability that this situation will occur means greater capacity-reducing effects of the major-street left-turning traffic on all Rank 3 movements.

The movement capacity $c_{m,k}$ for all Rank 3 movements is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor is denoted by f_k for all movements k and for all Rank 3 movements and is given by Equation 20-46.

Equation 20-46

$$f_k = \prod_j p_{0,j}$$

where Π indicates the product of a series of terms, and

$p_{0,j}$ = probability that conflicting Rank 2 movement j will operate in a queue-free state, and

k = Rank 3 movements.

The movement capacity $c_{m,k}$ for Rank 3 minor-street movements is computed with Equation 20-47.

$$c_{m,k} = c_{p,k} \times f_k$$

Equation 20-47

where $c_{p,k}$ is the potential capacity of Rank 3 minor-street movements, and f_k is the capacity adjustment factor that accounts for the impeding effects of higher-ranked movements computed according to Equation 20-46.

Step 8b: Rank 3 Capacity for Two-Stage Movements

If the Rank 3 movement is a two-stage movement, the procedure for computing the total movement capacity for the subject movement considering the two-stage gap acceptance process is as follows. An adjustment factor a and an intermediate variable y are computed with Equation 20-48 and Equation 20-49, respectively.

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} \quad \text{for } n_m > 0$$

Equation 20-48

$$y = \frac{c_{\mathrm{I}} - c_{m,x}}{c_{\mathrm{II}} - v_L - c_{m,x}}$$

Equation 20-49

where

n_m = number of vehicles that can be stored in the median;

c_{I} = movement capacity for the Stage I process (veh/h);

c_{II} = movement capacity for the Stage II process (veh/h);

v_L = major left-turn or U-turn flow rate, either $v_1 + v_{1U}$ or $v_4 + v_{4U}$ (veh/h); and

$c_{m,x}$ = capacity of subject movement, considering the total conflicting flow rate for both stages of a two-stage gap acceptance process (from Step 8a).

The total capacity c_T for the subject movement, considering the two-stage gap acceptance process, is computed by using Equation 20-50 and Equation 20-51 and incorporating the adjustment factors derived from Equation 20-48 and Equation 20-49.

For $y \neq 1$:

$$c_T = \frac{a}{y^{n_m+1} - 1}\left[y(y^{n_m} - 1)(c_{\mathrm{II}} - v_L) + (y - 1)c_{m,x}\right]$$

Equation 20-50

For $y = 1$:

$$c_T = \frac{a}{n_m + 1}\left[n_m(c_{\mathrm{II}} - v_L) + c_{m,x}\right]$$

Equation 20-51

Step 9: Compute Rank 4 Movement Capacities

Rank 4 movements occur only at four-leg intersections. Rank 4 movements (i.e., only the minor-street left turns at a four-leg intersection) can be impeded by all higher-ranked movements (Ranks 1, 2, and 3).

Step 9a: Rank 4 Capacity for One-Stage Movements

The probability that higher-ranked traffic movements will operate in a queue-free state is central to determining their overall impeding effects on the minor-street left-turn movement. However, not all these probabilities are independent of each other. Specifically, queuing in the major-street left-turning movement affects the probability of a queue-free state in the minor-street crossing movement. Applying the simple product of these two probabilities will likely overestimate the impeding effects on the minor-street left-turning traffic.

Exhibit 20-16 can be used to adjust for the overestimate caused by the statistical dependence between queues in streams of Ranks 2 and 3. The mathematical representation of this curve is determined with Equation 20-52.

Equation 20-52

$$p' = 0.65p'' - \frac{p''}{p''+3} + 0.6\sqrt{p''}$$

where

p' = adjustment to the major-street left, minor-street through impedance factor;

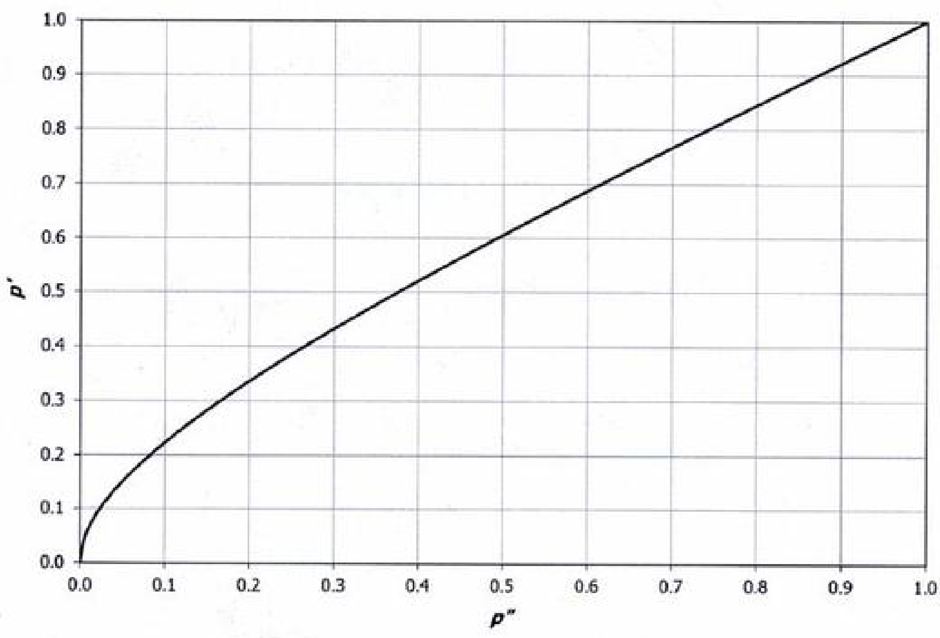
p'' = $(p_{0,j})(p_{0,k})$;

$p_{0,j}$ = probability of a queue-free state for the conflicting major-street left-turning traffic; and

$p_{0,k}$ = probability of a queue-free state for the conflicting minor-street crossing traffic.

When determining p' for Rank 4, Movement 7, in Equation 20-52, $p'' = (p_{0,1})(p_{0,4})(p_{0,11})$. Likewise, when determining p' for Rank 4, Movement 10, $p'' = (p_{0,1})(p_{0,4})(p_{0,8})$.

Exhibit 20-16
Adjustment to Impedance Factors for Major-Street Left-Turn Movement and Minor-Street Crossing Movement



The movement capacity $c_{m,l}$ for all Rank 4 movements is found by first computing a capacity adjustment factor that accounts for the impeding effects of higher-ranked movements. The capacity adjustment factor $f_{p,l}$ for the Rank 4 minor-street left-turn movement can be computed with Equation 20-53.

$$f_{p,l} = p' \times p_{0,j}$$

Equation 20-53

where

l = minor-street left-turn movement of Rank 4 (Movements 7 and 10 in Exhibit 20-1), and

j = conflicting Rank 2 minor-street right-turn movement (Movements 9 and 12 in Exhibit 20-1).

Finally, the movement capacity for the minor-street left-turn movements of Rank 4 is determined with Equation 20-54, where $f_{p,l}$ is the capacity adjustment factor that accounts for the impeding effects of higher-ranked movements.

$$c_{m,l} = c_{p,l} \times f_{p,l}$$

Equation 20-54

Step 9b: Rank 4 Capacity for Two-Stage Movements

The procedure for computing the total movement capacity for the subject movement considering the two-stage gap acceptance process is as follows. An adjustment factor a and an intermediate variable y are computed with Equation 20-55 and Equation 20-56, respectively.

$$a = 1 - 0.32e^{-1.3\sqrt{n_m}} \quad \text{for } n_m > 0$$

Equation 20-55

$$y = \frac{c_{\mathrm{I}} - c_{m,x}}{c_{\mathrm{II}} - v_L - c_{m,x}}$$

Equation 20-56

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 20-57 and Equation 20-58 and incorporating the adjustment factors computed in Equation 20-55 and Equation 20-56.

For $y \neq 1$:

$$c_T = \frac{a}{y^{n_m+1} - 1}\left[y(y^{n_m} - 1)(c_{\mathrm{II}} - v_L) + (y - 1)c_{m,x}\right]$$

Equation 20-57

For $y = 1$:

$$c_T = \frac{a}{n_m + 1}\left[n_m(c_{\mathrm{II}} - v_L) + c_{m,x}\right]$$

Equation 20-58

Step 10: Final Capacity Adjustments

Step 10a: Shared-Lane Capacity of Minor-Street Approaches

Where two or more movements share the same lane and cannot stop side by side at the stop line, Equation 20-59 is used to compute shared-lane capacity.

Equation 20-59

$$c_{SH} = \frac{\sum_y v_y}{\sum_y \frac{v_y}{c_{m,y}}}$$

where

c_{SH} = capacity of the shared lane (veh/h),

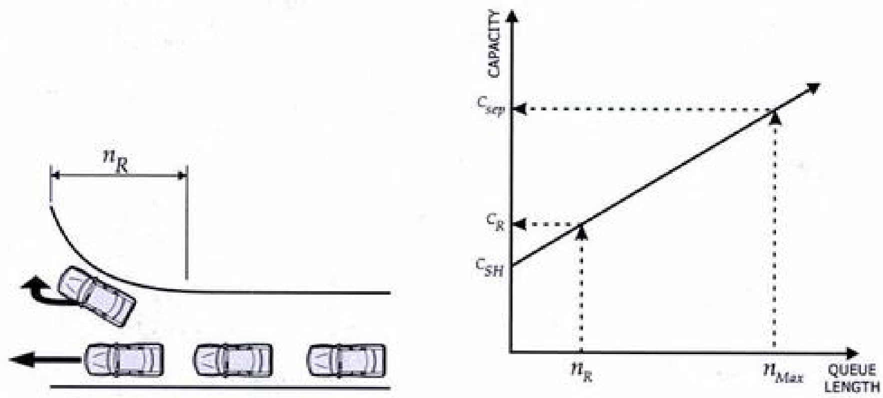
v_y = flow rate of the y movement in the subject shared lane (veh/h), and

$c_{m,y}$ = movement capacity of the y movement in the subject shared lane (veh/h).

Step 10b: Flared Minor-Street Lane Effects

To estimate the capacity of a flared right-turn lane (such as in Exhibit 20-17), the average queue length for each movement sharing the right lane on the minor-street approach must first be computed.

Exhibit 20-17
Capacity of a Flared-Lane Approach



This computation assumes the right-turn movement operates in one lane, and the other traffic in the right lane (upstream of the flare) operates in another, separate lane, as shown by Equation 20-60.

Equation 20-60

$$Q_{sep} = \frac{d_{sep} v_{sep}}{3{,}600}$$

where

Q_{sep} = average queue length for the movement considered as a separate lane (veh),

d_{sep} = control delay for the movement considered as a separate lane (as described in Step 11), and

v_{sep} = flow rate for the movement (veh/h).

Next, the required length of the storage area such that the approach would operate effectively as separate lanes is computed with Equation 20-61. This value is the maximum value of the queue lengths computed for each separate movement plus one vehicle.

$$n_{Max} = \max_i\left[\text{round}\left(Q_{sep,i} + 1\right)\right]$$

Equation 20-61

where

n_{Max} = length of the storage area such that the approach would operate as separate lanes;

$Q_{sep,i}$ = average queue length for movement i considered as a separate lane; and

round = round-off operator, rounding the quantity in parentheses to the nearest integer.

Next, the capacity of a separate lane condition c_{sep} must be computed and is assumed to be the capacity of right-turning traffic operating as a separate lane and the capacity of the other traffic in the right lane (upstream of the flare) operating as a separate lane. The capacity of a separate lane condition is calculated according to Equation 20-62.

$$c_{sep} = \min\left[c_R\left(1 + \frac{v_{L+TH}}{v_R}\right), c_{L+TH}\left(1 + \frac{v_R}{v_{L+TH}}\right)\right]$$

Equation 20-62

where

c_{sep} = sum of the capacity of the right-turning traffic operating as a separate lane and the capacity of the other traffic in the right lane (upstream of the flare) operating in a separate lane (veh/h),

c_R = capacity of the right-turn movement (veh/h),

c_{L+TH} = capacity of the through and left-turn movements as a shared lane (veh/h),

v_R = right-turn movement flow rate (veh/h), and

v_{L+TH} = through and left-turn movement combined flow rate (veh/h).

Finally, the capacity of the lane is computed, taking into account the flare. The capacity is interpolated as shown in Exhibit 20-17. A straight line is established by using the values of two points: (c_{sep}, n_{Max}) and (c_{SH}, 0). The interpolated value of the actual value of the flared-lane capacity c_R is computed with Equation 20-63.

$$c_R = \begin{cases} \left(c_{sep} - c_{SH}\right)\dfrac{n_R}{n_{Max}} + c_{SH} & \text{if } n_R \leq n_{Max} \\ c_{sep} & \text{if } n_R > n_{Max} \end{cases}$$

Equation 20-63

where

c_R = actual capacity of the flared lane (veh/h),

c_{sep} = capacity of the lane if both storage areas were infinitely long (refer to Equation 20-62) (veh/h),

c_{SH} = capacity of the lane when all traffic shares one lane (veh/h), and

n_R = actual storage area for right-turning vehicles as defined in Exhibit 20-17.

The actual capacity c_R must be greater than c_{SH} but less than or equal to c_{sep}.

Step 11: Compute Movement Control Delay

The delay experienced by a motorist is related to factors such as control type, geometrics, traffic, and incidents. In the TWSC intersection methodology, only that portion of delay attributed to the STOP-control aspect of the intersection, referred to as control delay, is quantified.

Control delay includes delay due to deceleration to a stop at the back of the queue from free-flow speed, move-up time within the queue, stopped delay at the front of the queue, and delay due to acceleration back to free-flow speed. With respect to field measurements, control delay is defined as the total time that elapses from the time a vehicle stops at the end of the queue to the time the vehicle departs from the stop line. This total elapsed time includes the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position, including deceleration of the vehicle from free-flow speed to the speed of vehicles in the queue.

Step 11a: Compute Control Delay to Rank 2 Through Rank 4 Movements

Average control delay for any particular minor movement is a function of the capacity of the approach and the degree of saturation. The analytical model used to estimate control delay (Equation 20-64) assumes demand is less than capacity for the period of analysis. If the degree of saturation is greater than about 0.9, average control delay is significantly affected by the length of the analysis period. In most cases, the recommended analysis period is 15 min. If demand exceeds capacity during a 15-min period, the delay results computed by the procedure may not be accurate. In this case, the period of analysis should be lengthened to include the period of oversaturation.

Equation 20-64

$$d = \frac{3{,}600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}}\right] + 5$$

where

d = control delay (s/veh),

v_x = flow rate for movement x (veh/h),

$c_{m,x}$ = capacity of movement x (veh/h), and

T = analysis time period (0.25 h for a 15-min period) (h).

A constant value of 5 s/veh is used to reflect delay during deceleration to and acceleration from a stop.

The constant 5 s/veh is included in Equation 20-64 to account for the deceleration of vehicles from free-flow speed to the speed of vehicles in the queue and the acceleration of vehicles from the stop line to free-flow speed.

Step 11b: Compute Control Delay to Rank 1 Movements

The effect of a shared lane on the major-street approach where left-turning vehicles may block Rank 1 through or right-turning vehicles can be significant. If no exclusive left-turn pocket is provided on the major street, a delayed left-turning vehicle may block the Rank 1 vehicles behind it. This will delay not only Rank 1 vehicles but also lower-ranked movements. While the delayed Rank 1 vehicles are discharging from the queue formed behind a left-turning vehicle, they impede lower-ranked conflicting movements.

Field observations have shown that such a blockage effect is usually very small, because the major street usually provides enough space for the blocked Rank 1 vehicle to bypass the left-turning vehicle on the right. At a minimum, incorporating this effect requires estimating the proportion of Rank 1 vehicles being blocked and computing the average delay to the major-street left-turning vehicles that are blocking through vehicles.

In the simplest procedure, the proportion of Rank 1 major-street vehicles not being blocked (i.e., in a queue-free state) is given by $p^*_{0,j}$ in Equation 20-43 ($p^*_{0,j}$ should be substituted for the major left-turn factor $p_{0,j}$ in Equation 20-43 in computing the capacity of lower-ranked movements that conflict). Therefore, the proportion of Rank 1 vehicles being blocked is $1 - p^*_{0,j}$.

The average delay to Rank 1 vehicles is computed with Equation 20-65.

$$d_{Rank1} = \begin{cases} \dfrac{(1 - p^*_{0,j}) d_{M,LT} \left(\frac{v_{i,1}}{N}\right)}{v_{i,1} + v_{i,2}} & N > 1 \\ (1 - p^*_{0,j}) d_{M,LT} & N = 1 \end{cases}$$

Equation 20-65

where

d_{Rank1} = delay to Rank 1 vehicles (s/veh),

N = number of through lanes per direction on the major street,

$p^*_{0,j}$ = proportion of Rank 1 vehicles not blocked (from Equation 20-43),

$d_{M,LT}$ = delay to major-street left-turning vehicles (from Equation 20-64) (s/veh),

$v_{i,1}$ = major-street through vehicles in shared lane (veh/h), and

$v_{i,2}$ = major-street turning vehicles in shared lane (veh/h).

On a multilane road, only the major-street volumes in the lane that may be blocked should be used in the computation as $v_{i,1}$ and $v_{i,2}$. On multilane roads, if it is assumed blocked Rank 1 vehicles do not bypass the blockage by moving into other through lanes (a reasonable assumption under conditions of high major-street flows), then $v_{i,1} = v_2/N$. Because of the unique characteristics associated with each site, the decision on whether to account for this effect is left to the analyst.

Step 12: Compute Approach and Intersection Control Delay

The control delay for all vehicles on a particular approach can be computed as the weighted average of the control delay estimates for each movement on the approach. Equation 20-66 is used for the computation.

$$d_A = \frac{d_r v_r + d_t v_t + d_l v_l}{v_r + v_t + v_l}$$

Equation 20-66

where

d_A = control delay on the approach (s/veh);

d_r, d_t, d_l = computed control delay for the right-turn, through, and left-turn movements, respectively (s/veh); and

v_r, v_t, v_l = volume or flow rate of right-turn, through, and left-turn traffic on the approach, respectively (veh/h).

Similarly, the intersection control delay d_I can be computed with Equation 20-67.

Equation 20-67

$$d_I = \frac{d_{A,1}v_{A,1} + d_{A,2}v_{A,2} + d_{A,3}v_{A,3} + d_{A,4}v_{A,4}}{v_{A,1} + v_{A,2} + v_{A,3} + v_{A,4}}$$

where $d_{A,x}$ is the control delay on approach x (s/veh), and $v_{A,x}$ is the volume or flow rate on approach x (veh/h).

In applying Equation 20-66 and Equation 20-67, the delay for all Rank 1 major-street movements is assumed to be 0 s/veh. LOS is not defined for an overall intersection because major-street movements with 0 s of delay typically result in a weighted average delay that is extremely low. As such, total intersection control delay calculations are typically used only when comparing control delay among different types of traffic control, such as two-way STOP control versus all-way STOP control.

Step 13: Compute 95th Percentile Queue Lengths

Queue length is an important consideration at unsignalized intersections. Theoretical studies and empirical observations have demonstrated that the probability distribution of queue lengths for any minor movement at an unsignalized intersection is a function of the capacity of the movement and the volume of traffic being served during the analysis period. Equation 20-68 can be used to estimate the 95th percentile queue length for any minor movement at an unsignalized intersection during the peak 15-min period on the basis of these two parameters (*10*).

Equation 20-68

$$Q_{95} \approx 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2 + \frac{\left(\frac{3{,}600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{150T}}\right]\left(\frac{c_{m,x}}{3{,}600}\right)$$

where

Q_{95} = 95th percentile queue (veh),

v_x = flow rate for movement x (veh/h),

$c_{m,x}$ = capacity of movement x (veh/h), and

T = analysis time period (0.25 h for a 15-min period) (h).

The mean queue length is computed as the product of the average delay per vehicle and the flow rate for the movement of interest. The expected total delay (vehicle hours per hour) equals the expected number of vehicles in the average queue; that is, the total hourly delay and the average queue are numerically identical. For example, four vehicle hours per hour of delay can be used interchangeably with an average queue length of four vehicles during the hour.

4. EXTENSION TO THE MOTORIZED VEHICLE METHODOLOGY

INTRODUCTION

This section presents the details of incorporating pedestrian effects on motorized vehicle capacity into the motorized vehicle methodology. The steps below replace Steps 6 through 9 from Section 3.

REPLACEMENT STEPS TO INCORPORATE PEDESTRIAN EFFECTS ON MOTORIZED VEHICLE CAPACITY

Step 6: Compute Rank 1 Movement Capacities

Rank 1 major-street movements are assumed to be unimpeded by any movements of lower rank. Major-street movements of Rank 1 are not expected to incur delay or slowing as they travel through the TWSC intersection. Empirical observations have shown that such delays occasionally occur, and they are accounted for by using adjustments provided later in this procedure.

For the purposes of this procedure, major-street movements of Rank 1 are assumed to be unimpeded by pedestrians at a TWSC intersection, even though research indicates some degree of Rank 1 vehicular yielding to pedestrians (see the pedestrian methodology in Section 5). The assumption that pedestrians do not impede Rank 1 major-street movements is a known limitation in the procedure.

Step 7: Compute Rank 2 Movement Capacities

Movements of Rank 2 (left turns from the major street and right turns from the minor street) must yield to conflicting major-street through and right-turning vehicular movements of Rank 1 as well as conflicting pedestrian movements of Rank 1. The movement capacity of each Rank 2 movement is equal to its potential capacity, factored by any impedance due to pedestrians.

Step 7a: Pedestrian Impedance

Minor vehicular movements must yield to conflicting pedestrian movements at a TWSC intersection. A factor accounting for pedestrian blockage is computed by Equation 20-69 on the basis of pedestrian volume, pedestrian walking speed, and width of the lane the minor movement is negotiating into.

$$f_{pb} = \frac{v_x \times \frac{w}{S_p}}{3{,}600}$$

Equation 20-69

where

f_{pb} = pedestrian blockage factor or proportion of time that one lane on an approach is blocked during 1 h;

v_x = number of groups of pedestrians, where x is Movement 13, 14, 15, or 16;

w = width of the lane the minor movement is negotiating into (ft); and

S_p = pedestrian walking speed, assumed to be 3.5 ft/s.

The pedestrian impedance factor for pedestrian movement x, $p_{p,x}$, is computed by Equation 20-70.

Equation 20-70

$$p_{p,x} = 1 - f_{pb}$$

Exhibit 20-18 shows that Rank 2 movements v_1 and v_4 must yield to pedestrian movements v_{16} and v_{15}, respectively. Exhibit 20-18 also shows that Rank 2 movement v_9 must yield to pedestrian movements v_{15} and v_{14}, and Rank 2 movement v_{12} must yield to pedestrian movements v_{16} and v_{13}. Rank 2 U-turn movements v_{1U} and v_{4U} are assumed to not yield to pedestrians crossing the major street, consistent with the assumptions stated previously for Rank 1 vehicles.

Exhibit 20-18
Relative Pedestrian–Vehicle Hierarchy for Rank 2 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_1	v_{16}	$p_{p,16}$
v_{1U}	—	—
v_4	v_{15}	$p_{p,15}$
v_{4U}	—	—
v_9	v_{15}, v_{14}	$(p_{p,15})(p_{p,14})$
v_{12}	v_{16}, v_{13}	$(p_{p,16})(p_{p,13})$

Step 7b: Movement Capacity for Major-Street Left-Turn Movements

Rank 2 major-street left-turn movements can be impeded by conflicting pedestrians. The movement capacity $c_{m,j}$ for major-street left-turn movements is computed with Equation 20-71.

Equation 20-71

$$c_{m,j} = c_{p,j} \times p_{p,i}$$

where j denotes movements of Rank 2 priority, i denotes movements of Rank 1 priority, and $c_{p,j}$ is the potential capacity of movement j.

Step 7c: Movement Capacity for Minor-Street Right-Turn Movements

The movement capacity $c_{m,j}$ for Rank 2 minor-street right-turn Movements 9 and 12 is impeded by two conflicting pedestrian movements. The capacity adjustment factors are denoted by f_9 and f_{12} for minor-street right-turn Movements 9 and 12, respectively, and are given by Equation 20-72 and Equation 20-73, respectively.

Equation 20-72

$$f_9 = p_{p,15} \times p_{p,14}$$

Equation 20-73

$$f_{12} = p_{p,16} \times p_{p,13}$$

where

f_9, f_{12} = capacity adjustment factor for Rank 2 minor-street right-turn Movements 9 and 12, respectively; and

$p_{p,j}$ = probability that conflicting Rank 2 pedestrian movement j will operate in a queue-free state.

The movement capacity for minor-street right-turn movements is then computed with Equation 20-74.

Equation 20-74

$$c_{m,j} = c_{p,j} \times f_j$$

where

$c_{m,j}$ = movement capacity for Movements 9 and 12,

$c_{p,j}$ = potential capacity for Movements 9 and 12 (from Step 5), and

f_j = capacity adjustment factor for Movements 9 and 12.

Step 7d: Movement Capacity for Major-Street U-Turn Movements

This step is the same as Step 7c in Section 3.

Step 7e: Effect of Major-Street Shared Through and Left-Turn Lane

This step is the same as Step 7d in Section 3.

Step 8: Compute Rank 3 Movement Capacities

Rank 3 minor-street traffic movements (minor-street through movements at four-leg intersections and minor-street left turns at three-leg intersections) must yield to conflicting Rank 1 and Rank 2 movements. Not all gaps of acceptable length that pass through the intersection are normally available for use by Rank 3 movements because some of them are likely to be used by Rank 2 movements.

If the Rank 3 movement is a two-stage movement, the movement capacity for the one-stage movement is computed as an input to the two-stage calculation.

Step 8a: Pedestrian Impedance

Exhibit 20-19 shows that Rank 3 movements v_8 and v_{11} must yield to pedestrian movements v_{15} and v_{16}.

Exhibit 20-19 Relative Pedestrian–Vehicle Hierarchy for Rank 3 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_8	v_{15}, v_{16}	$(p_{p,15})(p_{p,16})$
v_{11}	v_{15}, v_{16}	$(p_{p,15})(p_{p,16})$

The pedestrian impedance factor for Rank 3 movements is computed according to Equation 20-69 and Equation 20-70.

Step 8b: Rank 3 Capacity for One-Stage Movements

This step is the same as Step 8a in Section 3, except that the capacity adjustment factor f_k for all movements k and for all Rank 3 movements is given by Equation 20-75.

$$f_k = \prod_j p_{0,j} \times p_{p,x}$$

Equation 20-75

where Π indicates the product of a series of terms, and

$p_{0,j}$ = probability that conflicting Rank 2 movement j will operate in a queue-free state,

$p_{p,x}$ = probability of pedestrian movements of Rank 1 or Rank 2 priority,

k = Rank 3 movements, and

x = 13, 14, 15, or 16 (pedestrian movements of both Rank 1 and Rank 2).

Step 8c: Rank 3 Capacity for Two-Stage Movements

This step is the same as Step 8b in Section 3.

Step 9: Compute Rank 4 Movement Capacities

Rank 4 movements (i.e., only the minor-street left turns at a four-leg intersection) can be impeded by all higher-ranked movements (Ranks 1, 2, and 3).

Step 9a: Pedestrian Impedance

Exhibit 20-20 shows that Rank 4 movement v_7 must yield to pedestrian movements v_{15} and v_{13}, and Rank 4 movement v_{10} must yield to pedestrian movements v_{16} and v_{14}.

Exhibit 20-20
Relative Pedestrian–Vehicle Hierarchy for Rank 4 Movements

Vehicular Movement (v_x)	Must Yield to Pedestrian Movement	Impedance Factor for Pedestrians ($p_{p,x}$)
v_7	v_{15}, v_{13}	$(p_{p,15})(p_{p,13})$
v_{10}	v_{16}, v_{14}	$(p_{p,16})(p_{p,14})$

The pedestrian impedance factor for Rank 4 movements is computed according to Equation 20-69 and Equation 20-70.

Step 9b: Rank 4 Capacity for One-Stage Movements

This step is the same as Step 9a in Section 3, except that the capacity adjustment factor for the Rank 4 minor-street left-turn movement can be computed by Equation 20-76.

Equation 20-76

$$f_l = p' \times p_{0,j} \times p_{p,x}$$

where

l = minor-street left-turn movement of Rank 4,

j = conflicting Rank 2 minor-street right-turn movement, and

$p_{p,x}$ = values shown in Equation 20-70 (the variable $p_{0,j}$ should be included only if movement j is identified as a conflicting movement).

Step 9c: Rank 4 Capacity for Two-Stage Movements

This step is the same as Step 9b in Section 3.

5. PEDESTRIAN MODE

SCOPE OF THE METHODOLOGY

This methodology applies to TWSC intersections and midblock crossings at which pedestrians cross up to four through lanes on the major street. It is applied through a series of steps requiring input data related to vehicle and pedestrian volumes, geometric conditions, and motorist yield rates to pedestrians.

Spatial and Temporal Limits

This section's methodology applies to pedestrian crossings across an uncontrolled approach of a TWSC intersection or at a midblock location. The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This methodology produces the following performance measures:

- Average pedestrian delay, and
- LOS based on average pedestrian delay.

Limitations of the Methodology

The pedestrian methodology's limitations differ from the limitations of the motorized vehicle mode because the methods were developed in separate research efforts. The pedestrian methodology does not apply to undivided streets with more than four lanes, although it can accommodate up to four lanes in each direction separated by a median. It does not account for interaction effects of upstream signalized intersections, and it assumes random arrivals on the major street and equal directional and lane distribution on the major street.

The methodology does not take into account pedestrian cross flows (i.e., pedestrian flows approximately perpendicular to and crossing another pedestrian stream), and it assumes the pedestrian will reach the crossing without delay from pedestrians traveling parallel to the major street. Under high pedestrian volumes, this assumption may not be reasonable.

The method is for steady state conditions (i.e., the demand and capacity conditions are constant during the analysis period); it is not designed to evaluate how fast or how often the facility transitions from one demand or capacity state to another.

Alternative Tool Considerations

This section offers a method for estimating the delay and LOS for pedestrians crossing a major street at a TWSC intersection or midblock location. Some simulation-based tools offer a more detailed treatment of the arrival and departure of vehicles and their interaction with pedestrians, but for most purposes the HCM procedure produces an acceptable approximation.

The identified limitations for this chapter are shown in Exhibit 20-21, along with the potential for improved treatment by alternative tools.

Exhibit 20-21
Limitations of the HCM TWSC Pedestrian Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Undivided streets with more than four lanes	Simulation tools may be able to accommodate larger lane configurations.
Effects of upstream intersections	Simulation tools can include an unsignalized intersection explicitly within a signalized arterial or network.
Pedestrian cross flows parallel to the major street that impede pedestrian crossings across the major street	Simulation tools that model pedestrian flows explicitly may be able to capture this effect.
Non–steady state conditions for demand and capacity	Most alternative tools provide for multiperiod variation of demand and, in some cases, capacity.

REQUIRED INPUT DATA AND SOURCES

Exhibit 20-22 lists the information necessary to apply the pedestrian methodology and suggests potential sources for obtaining these data. It also suggests default values for use when specific information is not available.

Exhibit 20-22
Required Input Data, Potential Data Sources, and Default Values for TWSC Pedestrian Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Number of lanes on the major street	Design plans, road inventory	Must be provided
Crosswalk length (ft)	Design plans, road inventory	Must be provided
Crosswalk width (ft)	Design plans, road inventory	Must be provided
Presence of a raised median to allow a two-stage crossing	Design plans, road inventory	Must be provided
Posted speed limit	Design plans, road inventory	Must be provided
	Demand Data	
Pedestrian flow rate (p/s)	Field data, modeling	Must be provided
Presence of pedestrian platooning	Field data, modeling	Must be provided
Conflicting vehicular flow rate (veh/s)	Field data, modeling	Must be provided
Average pedestrian walking speed (ft/s)	Field data	3.5 ft/s
Pedestrian start-up time and end clearance time (s)	Field data	3 s
Mean motorist yielding rate to pedestrians	Field data, literature	Must be provided[a]

Note: [a] Sample values from the literature are provided in this section (see Exhibit 20-24).

COMPUTATIONAL STEPS

The required steps are illustrated in Exhibit 20-23.

Exhibit 20-23
TWSC Pedestrian Methodology

Step 1: Identify Two-Stage Crossings

↓

Step 2: Determine Critical Headway

↓

Step 3: Estimate Probability of a Delayed Crossing

↓

Step 4: Calculate Average Delay to Wait for Adequate Gap

↓

Step 5: Estimate Delay Reduction due to Yielding Vehicles

↓

Step 6: Calculate Average Pedestrian Delay and Determine LOS

Step 1: Identify Two-Stage Crossings

When a raised median refuge island is available, pedestrians typically cross in two stages, similar to the two-stage movement described for motorized vehicles earlier in this chapter. Determination of whether a median refuge exists may require engineering judgment. The main issue to determine is whether pedestrians cross the traffic streams in one or two stages. When pedestrians cross in two stages, pedestrian delay should be estimated separately for each stage of the crossing by using the procedures described in Steps 2 through 6, separating the conflicting vehicular volume accordingly. 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 is similar to critical headway for motorized vehicles.

The procedure for estimating critical headway for pedestrians is similar to that described for motorized vehicles. The critical headway is the minimum time interval in seconds below which a pedestrian will not attempt to begin crossing the street. Pedestrians use their judgment to determine whether the available headway between conflicting vehicles is long enough for a safe crossing. If the available headway is greater than the critical headway, it is assumed the pedestrian will cross, but if the available headway is less than the critical headway, it is assumed the pedestrian will not cross.

For a single pedestrian, critical headway is computed with Equation 20-77.

Equation 20-77

$$t_c = \frac{L}{S_p} + t_s$$

where

t_c = critical headway for a single pedestrian (s),

S_p = average pedestrian walking speed (ft/s),

L = crosswalk length (ft), and

t_s = pedestrian start-up time and end clearance time (s).

Groups of pedestrians require computation of their spatial distribution.

If groups of pedestrians are observed crossing in the field (i.e., a platoon, or more than one pedestrian crossing at a time), then the spatial distribution of pedestrians should be computed with Equation 20-78. The spatial distribution of pedestrians represents the number of rows of pedestrians waiting to cross, with the first row in position to cross and subsequent rows lined behind the first row. If the crosswalk is wide enough to accommodate a group of pedestrians traveling side-by-side without needing to also travel behind one another, then the spatial distribution of pedestrians equals one. If no pedestrian grouping is observed, the spatial distribution of pedestrians is assumed to be one.

Equation 20-78

$$N_p = \text{int}\left[\frac{8.0(N_c - 1)}{W_c}\right] + 1$$

where

N_p = spatial distribution of pedestrians (p),

N_c = total number of pedestrians in the crossing platoon (from Equation 20-79) (p),

W_c = crosswalk width (ft), and

8.0 = default clear effective width used by a single pedestrian to avoid interference when passing other pedestrians (ft).

To compute spatial distribution, the analyst must make field observations or estimate the platoon size by using Equation 20-79.

Equation 20-79

$$N_c = \frac{v_p e^{v_p t_c} + v e^{-v t_c}}{(v_p + v) e^{(v_p - v) t_c}}$$

where

N_c = total number of pedestrians in the crossing platoon (p),

v_p = pedestrian flow rate (p/s),

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings), and

t_c = single pedestrian critical headway (s).

The group critical headway is the critical headway needed to accommodate a group of pedestrians. The group critical headway is determined with Equation 20-80.

Equation 20-80

$$t_{c,G} = t_c + 2(N_p - 1)$$

where

$t_{c,G}$ = group critical headway (s),

t_c = critical headway for a single pedestrian (s), and

N_p = spatial distribution of pedestrians (p).

Step 3: Estimate Probability of a Delayed Crossing

On the basis of the calculation of the critical headway $t_{c,G}$, the probability that a pedestrian will not incur any crossing delay is equal to the likelihood that a pedestrian will encounter a gap greater than or equal to the critical headway immediately upon arrival at the intersection.

Assuming random arrivals of vehicles on the major street and equal distribution of vehicles among all through lanes on the major street, the likelihood that a gap in a given lane does not exceed the critical headway is as shown in Equation 20-81. Because traffic is assumed to be distributed independently in each through lane, Equation 20-82 shows the probability that a pedestrian incurs nonzero delay at a TWSC crossing.

Equation 20-81

$$P_b = 1 - e^{\frac{-t_{c,G} v}{N_L}}$$

Equation 20-82

$$P_d = 1 - (1 - P_b)^{N_L}$$

where

P_b = probability of a blocked lane,

P_d = probability of a delayed crossing,

N_L = number of through lanes crossed,

$t_{c,G}$ = group critical headway (s), and

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings).

Step 4: Calculate Average Delay to Wait for Adequate Gap

Research indicates average delay to pedestrians at unsignalized crossings, assuming no motor vehicles yield and the pedestrian is forced to wait for an adequate gap, depends on the critical headway, the vehicular flow rate of the subject crossing, and the mean vehicle headway (*11*). The average delay per pedestrian to wait for an adequate gap is given by Equation 20-83.

$$d_g = \frac{1}{v}\left(e^{vt_{c,G}} - vt_{c,G} - 1\right)$$

Equation 20-83

where

d_g = average pedestrian gap delay (s),

$t_{c,G}$ = group critical headway (s), and

v = conflicting vehicular flow rate (veh/s) (combined flows for one-stage crossings; separate flows for two-stage crossings).

The average delay for any pedestrian who is unable to cross immediately upon reaching the intersection (e.g., any pedestrian experiencing nonzero delay) is thus a function of P_d and d_g, as shown in Equation 20-84.

$$d_{gd} = \frac{d_g}{P_d}$$

Equation 20-84

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 20-83 estimates pedestrian delay when motorists on the major approaches do not yield to pedestrians. When motorist yield rates are significantly higher than zero, pedestrians will experience considerably less delay than that estimated by Equation 20-83.

In the United States, motorists are legally required to yield to pedestrians, under most circumstances, in both marked and unmarked crosswalks. However, actual motorist yielding behavior varies considerably. 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 (*12, 13*) provides information on motorist responses to typical pedestrian crossing treatments, as shown in Exhibit 20-24. The exhibit shows results from two 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 20-24 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 20-24
Effect of Pedestrian Crossing Treatments on Motorist Yield Rates

	Staged Pedestrians		Unstaged Pedestrians	
Crossing Treatment	**No. of Sites**	**Mean Yield Rate (%)**	**No. 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	NA	NA	17	81

Note: NA = not available.
Source: Fitzpatrick et al. (*12*) and Shurbutt et al. (*13*).

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. 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 vehicles do not yield, 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.

Average pedestrian delay can be calculated with Equation 20-85, where the first term in the equation represents expected delay from crossings occurring when motorists yield, and the second term represents expected delay from crossings when pedestrians wait for an adequate gap.

Equation 20-85

$$d_p = \sum_{i=1}^{n} h(i - 0.5)P(Y_i) + \left(P_d - \sum_{i=1}^{n} P(Y_i)\right) d_{gd}$$

where

d_p = average pedestrian delay (s);

i = crossing event (i = 1 to n);

h = average headway for each through lane = (N_L/v) (s);

$P(Y_i)$ = probability that motorists yield to pedestrian on crossing event i;

P_d = probability of a delayed crossing; and

n = average number of crossing events before an adequate gap is available = int(d_{gd}/h).

Equation 20-85 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 20-86 gives $P(Y_i)$ for any i.

$$P(Y_i) = P_d M_y \left(1 - M_y\right)^{i-1}$$

Equation 20-86

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 given by Equation 20-87.

$$P(Y_1) = 2P_b(1 - P_b)M_y + P_b^2 M_y^2$$

Equation 20-87

where P_b is the probability of a blocked lane.

Equation 20-88 shows $P(Y_i)$ where i is greater than one. Equation 20-88 is equivalent to Equation 20-87 if $P(Y_0)$ is set to equal zero.

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{(2P_b[1 - P_b]M_y) + (P_b^2 M_y^2)}{P_d}\right]$$

Equation 20-88

Three-Lane Crossing

A three-lane crossing follows the same principles as a two-lane crossing. Equation 20-89 shows the calculation for $P(Y_i)$.

Equation 20-89

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right]\left[\frac{P_b^3 M_y^3 + 3P_b^2(1-P_b)M_y^2 + 3P_b(1-P_b)^2 M_y}{P_d}\right]$$

where $P(Y_0) = 0$.

Four-Lane Crossing

A four-lane crossing follows the same principles as above. Equation 20-90 shows the calculation for $P(Y_i)$.

Equation 20-90

$$P(Y_i) = \left[P_d - \sum_{j=0}^{i-1} P(Y_j)\right] \times \left[\frac{P_b^4 M_y^4 + 4P_b^3(1-P_b)M_y^3 + 6P_b^2(1-P_b)^2 M_y^2 + 4P_b(1-P_b)^3 M_y}{P_d}\right]$$

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 20-3 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.

6. BICYCLE MODE

As of the publication of this edition of the HCM, no methodology specific to bicyclists has been developed to assess the performance of bicyclists at TWSC intersections, as few data are available in the United States to support model calibration or LOS definitions. Depending on individual comfort level, ability, geometric conditions, and traffic conditions, a bicyclist may travel through the intersection either as a motor vehicle or as a pedestrian. Critical headway distributions have been identified in the research (*14, 15*) for bicycles crossing two-lane major streets. Data on critical headways for bicycles under many circumstances are not readily available, however. Bicycles also differ from motor vehicles in that they normally do not queue linearly at a STOP sign. Instead, multiple bicycles often use the same gap in the vehicular traffic stream. This practice probably affects the determination of bicycle follow-up time. This phenomenon and others described in this section have not been adequately researched and are not explicitly included in the methodology.

7. APPLICATIONS

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary engineering analysis.

Operational Analysis

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement in vehicles per hour, percentage of heavy vehicles for each approach, peak hour factor for all demand volumes, lane configurations, specific geometric conditions, and upstream signal information. The outputs of an operational analysis are estimates of capacity, control delay, and queue lengths. The steps of the methodology, described in this chapter's methodology section, are followed directly without modification.

Design Analysis

The operational analysis described earlier in this chapter can be used for design purposes by using a given set of traffic flow data and iteratively determining the number and configuration of lanes that would be required to produce a given LOS.

Planning and Preliminary Engineering Analysis

The operational analysis method described earlier in this chapter provides a detailed procedure for evaluating the performance of a TWSC intersection. To estimate LOS for a future time horizon, a planning analysis based on the operational method is used. The planning method uses all the geometric and traffic flow data required for an operational analysis, and the computations are identical. However, input variables for percentage of heavy vehicles and peak hour factor are typically estimated (or defaults are used) when planning applications are performed.

EXAMPLE PROBLEMS

Section 2 of Chapter 32, STOP-Controlled Intersections: Supplemental, provides five example problems that illustrate each of the computational steps involved in applying the motorized vehicle method:

1. Analyze a TWSC intersection with three legs,
2. Analyze a pedestrian crossing at a TWSC intersection,
3. Analyze a TWSC intersection with flared approaches and median storage,
4. Analyze a TWSC intersection within a signalized urban street segment, and
5. Analyze a TWSC intersection on a six-lane street with U-turns and pedestrians.

EXAMPLE RESULTS

Analysis of TWSC intersections is commonly performed to determine whether an existing intersection or driveway can remain as a TWSC intersection or whether additional treatments are necessary. These treatments, including geometric modifications and changes in traffic control, are discussed in other references, including the presentation of traffic signal warrants in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (16). This section discusses two common situations analysts face: the analysis of shared versus separate lanes and the interpretation of LOS F.

Analysis of Shared Versus Separate Lanes

Interpretation of the effects of shared lanes should consider both delay associated with individual movements and delay associated with all vehicles on a given approach.

Some movements, most often left-turn movements, can sometimes have a poorer LOS when given a separate lane than when they share a lane with another movement (usually a through movement). This is not inconsistent in terms of the stated criteria. Left-turn movements will generally experience longer control delays than other movements because of the nature and priority of the movement. The control delay for left turns in a shared lane may be less than the control delay for left turns in a separate lane. However, if delay for all vehicles on the approach or at the intersection is considered, providing separate lanes will result in lower total delay.

Interpretation of LOS F

LOS F occurs when there are not enough gaps of suitable size to allow minor-street vehicles to enter or cross through traffic on the major street; this results in long average control delays (greater than 50 s/veh). Depending on the demand on the approach, long queues on the minor approaches may result. The method, however, is based on a constant critical headway.

LOS F may also appear in the form of drivers on the minor street selecting smaller-than-usual gaps. In such cases, safety issues may occur, and some disruption to the major traffic stream may result. With lower demands, LOS F may not always result in long queues.

At TWSC intersections, the critical movement, often the minor-street left turn, may control the overall performance of the intersection. The lower threshold for LOS F is set at 50 s of delay per vehicle. In some cases, the delay equations will predict delays greater than 50 s for minor-street movements under very low-volume conditions on the minor street (fewer than 25 veh/h). On the basis of the first term of the delay equation, the LOS F threshold is reached with a movement capacity of approximately 85 veh/h or less, regardless of the minor-street movement volume.

This analysis procedure assumes random arrivals on the major street. For a typical major street with two lanes in each direction and an average traffic volume in the range of 15,000 to 20,000 veh/day (roughly equivalent to a peak hour flow rate of 1,500 to 2,000 veh/h), the delay equation will predict greater than 50 s of delay (LOS F) for many urban TWSC intersections that allow minor-street left-turn movements. LOS F will be predicted regardless of the volume of minor-street left-turning traffic. Even with a LOS F estimate, most low-volume minor-street approaches would not meet any of the volume or delay warrants for signalization noted in the *Manual on Uniform Traffic Control Devices* (16). As a result, analysts who use the HCM LOS thresholds as the sole measure to determine the design adequacy of TWSC intersections should do so with caution.

In evaluating the overall performance of TWSC intersections, it is important to consider measures of effectiveness such as volume-to-capacity ratios for individual movements, average queue lengths, and 95th percentile queue lengths in addition to considering delay. By focusing on a single measure of effectiveness for the worst movement only, such as delay for the minor-street left turn, users may make less effective traffic control decisions.

8. REFERENCES

Some of these references are available in the Technical Reference Library in Volume 4.

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CHAPTER 21
ALL-WAY STOP-CONTROLLED INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

All-way STOP-controlled (AWSC) intersections are common in the United States. They are characterized by having all approaches controlled by STOP signs without any street having priority. Streets intersecting at AWSC intersections can be public streets or private driveways. This chapter presents concepts and procedures for analyzing these types of intersections (*1*). A glossary and list of symbols, including those used for AWSC intersections, is provided in Chapter 9.

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, although qualitative guidance is provided for analysis of pedestrians and bicyclists.

CHAPTER ORGANIZATION

This chapter is organized into the following sections:

- Section 1 (this section) introduces the chapter.
- Section 2 presents basic concepts of the AWSC methodology, including a description of the phase-pattern concept, general capacity concepts, and level-of-service (LOS) criteria.
- Section 3 describes the methodological details of the procedure, including a step-by-step description of the analysis steps, a discussion of limitations of the method, and required data.
- Section 4 addresses extensions of the analysis methodology for motor vehicles specifically related to analysis of three-lane AWSC intersection approaches.
- Section 5 and Section 6, respectively, present pedestrian and bicycle evaluation considerations for AWSC intersections.
- Section 7 describes example problems included in Volume 4, suggests applications for alternative tools, and provides guidance on interpreting analysis results.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 32, Stop-Controlled Intersections: Supplemental, provides example problems for the AWSC methodology and additional methodological details.
- Section M, STOP-Controlled Intersections, in the *Planning and Preliminary Engineering Applications Guide to the HCM* in Volume 4, provides guidance on analyzing AWSC intersections in the context of a planning study.

2. CONCEPTS

AWSC intersections are a type of unsignalized intersection that require drivers on all approaches 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.

Although 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. Drivers develop 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.

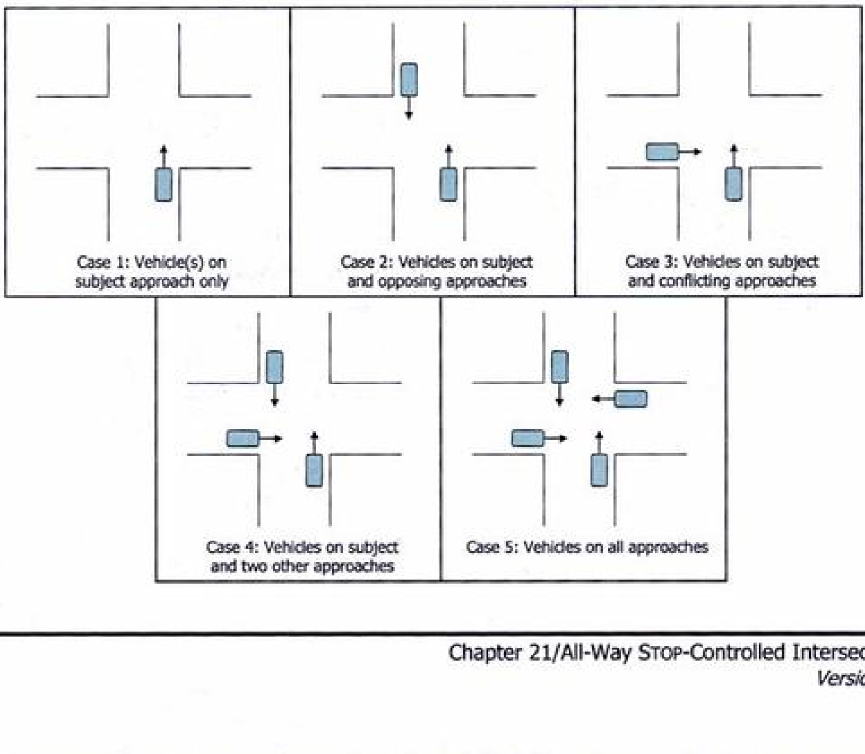
The methodology analyzes each intersection approach separately. The approach under study is called the subject approach. The opposing approach and the conflicting approaches create conflicts with vehicles on the subject approach.

PHASE PATTERNS

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 21-1.

Exhibit 21-1
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.

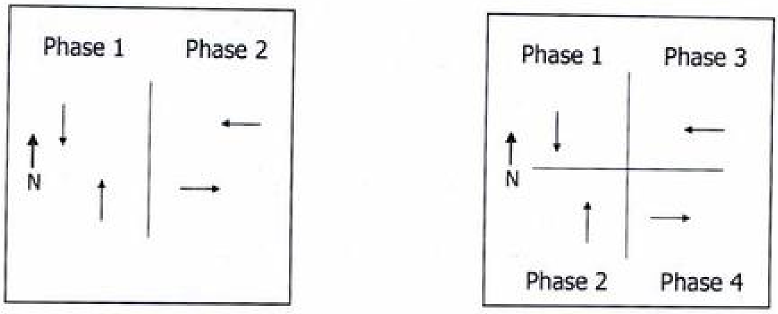
The capacity of an AWSC intersection can be determined as a function of saturation headway, departure headway, and service time.

Capacity is defined as the maximum throughput on an approach given the flow rates on the other intersection approaches. In general, the capacity at an AWSC intersection can be determined as a function of 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).

A two-phase pattern, as shown in Exhibit 21-2(a), is observed at a standard four-leg AWSC intersection (one approach lane on each leg), when 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 21-2(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 21-2
Operation Patterns at AWSC Intersections



(a) Two-phase (single-lane approaches) (b) Four-phase (multilane approaches)

DEPARTURE HEADWAY

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, drivers on the subject approach can enter the intersection immediately after stopping. However, if vehicles are waiting on a conflicting approach, a driver from the subject approach must wait for consensus with the next conflicting driver. 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

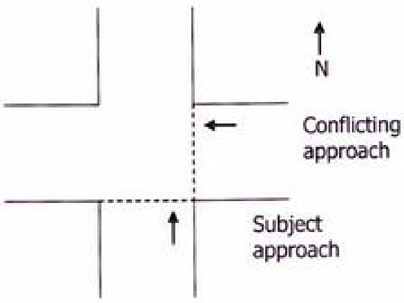
The capacity model described in more detail here is an expansion of earlier work (2). The model is described for four increasingly complex cases: the intersection of two one-way streets with no turning movements, the intersection of two two-way streets with no turning movements, a generalized model for 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 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 21-3.

The impact of turning movements is considered below as part of the generalized models.

Exhibit 21-3
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 21-1.

Equation 21-1

$$h_{d,N} = h_{s1}(1 - x_W) + h_{s2}x_W$$

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 21-2.

Equation 21-2

$$h_{d,W} = h_{s1}(1 - x_N) + h_{s2}x_N$$

Because 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 shown in Equation 21-3 and Equation 21-4.

Equation 21-3

$$h_{d,N} = \frac{h_{s1}[1 + \lambda_W(h_{s2} - h_{s1})]}{1 - \lambda_N\lambda_W(h_{s2} - h_{s1})^2}$$

Equation 21-4

$$h_{d,W} = \frac{h_{s1}[1 + \lambda_N(h_{s2} - h_{s1})]}{1 - \lambda_N\lambda_W(h_{s2} - h_{s1})^2}$$

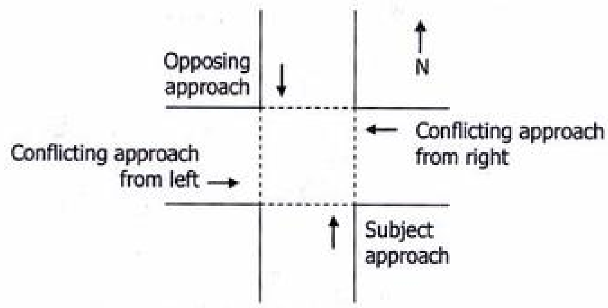
Intersection of Two Two-Way Streets

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, as shown in Exhibit 21-4. 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 21-5.

Equation 21-5

$$h_{d,N} = h_{s1}(1 - x_E)(1 - x_W) + h_{s2}[1 - (1 - x_E)(1 - x_W)]$$

Exhibit 21-4
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 21-5.

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 21-5 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, are 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 21-5
Probability of Degree-of-Conflict Case

Degree-of-Conflict Case	Opposing	Approach Conflicting Left	Conflicting Right	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	$(x_O)(x_{CL})(x_{CR})$

Notes: N = no; Y = yes.

The probability $P(C_i)$ for each degree-of-conflict case given in Exhibit 21-5 can be computed with Equation 21-6 through Equation 21-10. 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 21-6**

$$P(C_2) = (x_O)(1 - x_{CL})(1 - x_{CR})$$ **Equation 21-7**

$$P(C_3) = (1 - x_O)(x_{CL})(1 - x_{CR}) + (1 - x_O)(1 - x_{CL})(x_{CR})$$ **Equation 21-8**

$$P(C_4) = (x_O)(1 - x_{CL})(x_{CR}) + (x_O)(x_{CL})(1 - x_{CR}) + (1 - x_O)(x_{CL})(x_{CR})$$ **Equation 21-9**

$$P(C_5) = (x_O)(x_{CL})(x_{CR})$$ **Equation 21-10**

The departure headway for an approach is the expected value of the saturation headway distribution, computed by Equation 21-11.

$$h_d = \sum_{i=1}^{5} P(C_i) h_{si}$$ **Equation 21-11**

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.

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.

Capacity is determined by increasing volume on the subject approach until the degree of utilization exceeds 1.0 (i.e., x > 1.0).

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, increasing driver decision time and the saturation headway.

However, some movements may not conflict with each other to the same degree 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 above proposes 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 21-6 and Exhibit 21-7 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 21-6
Degree-of-Conflict Cases for Two-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	No. of Opposing and Conflicting Vehicles
1				0
2	x			1, 2
3		x 	 x	1, 2
4	x x 	x x	 x x	2, 3, 4
5	x	x	x	3, 4, 5, 6

Exhibit 21-7
Degree-of-Conflict Cases for Three-Lane Approaches

Degree-of-Conflict Case	Approaches with Vehicles: Opposing	Conflicting Left	Conflicting Right	No. of Opposing and Conflicting Vehicles
1				0
2	x			1, 2, 3
3		x 	 x	1, 2, 3
4	x x 	x x	 x x	2, 3, 4, 5, 6
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.

LEVEL-OF-SERVICE CRITERIA

LOS criteria for AWSC intersections are given in Exhibit 21-8. 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 21-8
LOS Criteria: Motorized Vehicle Mode

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a] $v/c \le 1.0$	$v/c > 1.0$
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.

3. MOTORIZED VEHICLE CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

This section focuses on the operation of AWSC intersections. This version of the AWSC intersection analysis procedures is primarily a result of studies conducted in National Cooperative Highway Research Program Project 3-46 (*1*).

Spatial and Temporal Limits

The methodology assumes the AWSC intersection under investigation is isolated. If interaction effects (e.g., queue spillback, demand starvation) are likely between the subject AWSC intersection and other intersections, the use of alternative tools may result in a more accurate analysis. Analysis boundaries may also include different demand scenarios related to time of day or to different development scenarios that produce different demand flow rates.

The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Limitations of the Methodology

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 motor vehicles 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.

Use of Alternative Tools

Except for the effects of interaction with other intersections, the limitations of the methodology stated above 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.

REQUIRED INPUT DATA AND SOURCES

Exhibit 21-9 lists the information necessary to apply the motor vehicle methodology and suggests potential sources for obtaining these data. It also suggests default values for use when intersection-specific information is not available.

A comprehensive presentation of potential default values for interrupted-flow facilities is available elsewhere (3). These defaults cover the key characteristics of peak hour factor and percentage of heavy vehicles. Recommendations are based on geographic region, population, and time of day. All general default values for interrupted-flow facilities may be applied to analysis of AWSC intersections in the absence of field data or projections of conditions.

Exhibit 21-9
Required Input Data, Potential Data Sources, and Default Values for AWSC Motor Vehicle Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
Geometric Data		
Number and configuration of lanes of each approach	Design plans, road inventory	Must be provided
Demand Data		
Hourly turning movement demand volume (veh/h) AND peak hour factor OR Hourly turning movement demand flow rate (veh/h)	Field data, modeling	Must be provided
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor (decimal)	Field data	0.92
Heavy-vehicle percentage (%)	Field data	3%

COMPUTATIONAL STEPS

The AWSC intersection methodology for the motor vehicle mode is applied through a series of steps that relate to input data, saturation headways, departure headways, service time, capacity, and LOS. The steps are listed in Exhibit 21-10.

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 to peak 15-min flow rates in vehicles per hour as given in Equation 21-12.

$$v_i = \frac{V_i}{PHF}$$

Equation 21-12

where

v_i = demand flow rate for movement i (veh/h),

V_i = demand volume for movement i (veh/h), and

PHF = intersection peak hour factor.

If PHF is used, a single intersectionwide PHF should be used rather than movement-specific or approach-specific PHFs. If individual approaches or movements peak at different times, a series of 15-min analysis periods that encompass the peaking should be considered.

The use of a peak 15-min traffic count multiplied by four is preferred for existing conditions when traffic counts are available. The use of a 1-h demand volume divided by a peak hour factor is preferred with projected volumes or with projected volumes that have been added to current volumes.

If peak hour factors are used, a single peak hour factor for the entire intersection is generally preferred to decrease the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, but in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. In the event individual approaches or movements are known to have substantially different peaking characteristics or peak during different 15-min periods within the hour, a series of 15-min analysis periods that encompass the peaking should be considered instead of a single analysis period using a single peak hour factor for the intersection.

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 21-14. 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 21-14

$$x = \frac{vh_d}{3{,}600}$$

Step 7: Compute Probability States

The probability state of each combination i is found with Equation 21-15.

Equation 21-15

$$P(i) = \prod_j P(a_j)$$

where Π represents the product of a series of terms, and

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 21-13, where V_j is the lane flow rate; and

a_j = 1 (indicating a vehicle present in the lane) or 0 (indicating no vehicle present in the lane) (values of a_j for each lane in each combination i are listed in Exhibit 21-14).

Exhibit 21-13
Probability of a_j (Two-Lane Approaches)

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 21-14.

Tables for three-lane approaches are given in Chapter 32, Stop-Controlled Intersections: Supplemental.

Exhibit 21-14 provides the 64 possible combinations when alternative lane occupancies are considered for two-lane approaches. A 1 indicates a vehicle is in the lane, and a 0 indicates 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 21-14
Probability of Degree-of-Conflict Case: Multilane AWSC Intersections (Two-Lane Approaches, by Lane)

			Opposing Approach		Conflicting Left Approach		Conflicting Right Approach	
i	DOC Case	No. of Vehicles	Lane 1	Lane 2	Lane 1	Lane 2	Lane 1	Lane 2
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

Notes: DOC = degree-of-conflict; No. of Vehicles = total number of vehicles on opposing and conflicting approaches.

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 21-16 through Equation 21-20 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 21-14).

Equation 21-16

$$P(C_1) = P(1)$$

Equation 21-17

$$P(C_2) = \sum_{i=2}^{4} P(i)$$

Equation 21-18

$$P(C_3) = \sum_{i=5}^{10} P(i)$$

Equation 21-19

$$P(C_4) = \sum_{i=11}^{37} P(i)$$

Equation 21-20

$$P(C_5) = \sum_{i=38}^{64} P(i)$$

The probability adjustment factors are then computed with Equation 21-21 through Equation 21-25.

Equation 21-21

$$AdjP(1) = \alpha[P(C_2) + 2P(C_3) + 3P(C_4) + 4P(C_5)]/1$$

Equation 21-22

$$AdjP(2) \text{ through } AdjP(4) = \alpha[P(C_3) + 2P(C_4) + 3P(C_5) - P(C_2)]/3$$

Equation 21-23

$$AdjP(5) \text{ through } AdjP(10) = \alpha[P(C_4) + 2P(C_5) - 3P(C_3)]/6$$

Equation 21-24

$$AdjP(11) \text{ through } AdjP(37) = \alpha[P(C_5) - 6P(C_4)]/27$$

Equation 21-25

$$AdjP(38) \text{ through } AdjP(64) = -\alpha[10P(C_5)]/27$$

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 21-26.

Equation 21-26

$$P'(i) = P(i) + AdjP(i)$$

Step 9: Compute Saturation Headways

The saturation headway h_{si} is the sum of the base saturation headway as presented in Exhibit 21-15 and the saturation headway adjustment factor from Step 4. It is shown in Equation 21-27.

Equation 21-27

$$h_{si} = h_{\text{base}} + h_{adj}$$

Exhibit 21-15
Saturation Headway Values by Case and Geometry Group

Case	No. of Vehicles	Base Saturation Headway, h_{base} (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

Step 10: Compute Departure Headways

The departure headway h_d of the lane is the expected value of the saturation headway distribution as given by Equation 21-28.

$$h_d = \sum_{i=1}^{64} P'(i) h_{si}$$

Equation 21-28

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 lane in a subject 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 lane on each approach until the degree of utilization for the subject lane reaches 1. When this degree of utilization occurs, the final value of the subject lane flow rate is the maximum possible throughput or capacity of this lane. The following steps illustrate this process.

Step 12a: Select a Subject Approach and Step 12b: Establish a Trial Volume for the Subject Approach

If the degree of utilization calculated for the subject approach is less than 1, then the trial volume for the subject approach should be increased. If the degree of utilization calculated for the subject approach is greater than 1, then the trial volume for the subject approach should be decreased.

Step 12c: Compute the Degree of Utilization Using Steps 5 Through 11 and Step 12d: Check the Degree of Utilization x

If the calculated degree of utilization x is not 1, return to Step 12b and use a different trial volume. When the degree of utilization equals 1, the trial volume that was selected is the capacity of the subject approach.

Step 12e: Repeat Steps 12a Through 12d for the Other Approaches

Step 13: Compute Service Times

The service time t_s required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time with Equation 21-29.

Equation 21-29

$$t_s = h_d - m$$

where 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 and Determine LOS for Each Lane

The delay experienced by a motorist is related to factors such as 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 21-30 can be used to compute control delay for each lane.

Equation 21-30

$$d = t_s + 900T\left[(x-1) + \sqrt{(x-1)^2 + \frac{h_d x}{450T}}\right] + 5$$

where

d = average control delay (s/veh),

x = $vh_d/3{,}600$ = degree of utilization (unitless),

t_s = service time (s),

h_d = departure headway (s), and

T = length of analysis period (h).

The LOS for each approach and the LOS for the intersection are determined by using the computed values of control delay, with Exhibit 21-8.

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 21-31.

Equation 21-31

$$d_a = \frac{\sum d_i v_i}{\sum v_i}$$

where

d_a = 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 21-32.

$$d_{\text{intersection}} = \frac{\sum d_a v_a}{\sum v_a}$$

Equation 21-32

where

$d_{\text{intersection}}$ = control delay for the entire intersection (s/veh),

d_a = control delay for approach a (s/veh), and

v_a = flow rate for approach a (veh/h).

The LOS for each approach and for the intersection are determined by using the computed values of control delay, with Exhibit 21-8.

Step 16: Compute Queue Lengths

Research (*4*) 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 21-33 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 21-33

where

Q_{95} = 95th percentile queue (veh),

x = $vh_d/3{,}600$ = degree of utilization (unitless),

h_d = departure headway (s), and

T = length of analysis period (h).

4. EXTENSION TO THE MOTORIZED VEHICLE METHODOLOGY

INTRODUCTION

This section provides details for a procedure to analyze three-lane approaches at AWSC intersections. This procedure is an extension of the methodology described in the previous section, and the same general capacity concepts apply.

REPLACEMENT STEPS FOR THREE-LANE AWSC INTERSECTION APPROACHES

The methodology for three-lane AWSC approaches is fundamentally the same as for two-lane approaches. The process used to determine geometry groups (Step 3) will typically result in Geometry Group 5 or 6 (per Exhibit 21-11), which affects the saturation headway adjustments in Step 4. Steps 7 and 8 increase in complexity to accommodate three-lane approaches and are replaced with the following steps. The remaining steps remain as presented in Section 3.

Step 7: Compute Probability States

The probability state of each combination i is found with Equation 21-34.

Equation 21-34

$$P(i) = \prod_{j} P(a_j)$$

where Π represents the product of a series of terms, and

j = O1 (opposing approach, Lane 1), O2 (opposing approach, Lane 2), O3 (opposing approach, Lane 3), CL1 (conflicting left approach, Lane 1), CL2 (conflicting left approach, Lane 2), CL3 (conflicting left approach, Lane 3), CR1 (conflicting right approach, Lane 1), CR2 (conflicting right approach, Lane 2), and CR3 (conflicting right approach, Lane 3) for a three-lane, two-way AWSC intersection;

$P(a_j)$ = probability of a_j, computed on the basis of Exhibit 21-16, where V_j is the lane flow rate; and

a_j = 1 (indicating a vehicle present in the lane) or 0 (indicating no vehicle present in the lane) (values of a_j for each lane in each combination i are listed in Exhibit 32-15 in Chapter 32, STOP-Controlled Intersections: Supplemental).

Exhibit 21-16
Probability of a_j (Three-Lane Approaches)

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 21-14.

Tables for three-lane approaches are given in Chapter 32, STOP-Controlled Intersections: Supplemental

For three-lane AWSC approaches, the number of possible combinations of degree-of-conflict cases when alternative lane occupancies are considered increases from 64 cases to 512 cases. These 512 cases are presented in tabular form in Exhibit 32-15 in Chapter 32, STOP-Controlled Intersections: Supplemental.

Step 8: Compute Probability Adjustment Factors

The probability adjustment is computed with Equation 21-35 through Equation 21-39 to account for the serial correlation in the previous probability computation. First, the probability of each degree-of-conflict case must be determined (assuming the 512 cases presented in Exhibit 32-15 in Chapter 32, STOP-Controlled Intersections: Supplemental).

$$P(C_1) = P(1)$$ **Equation 21-35**

$$P(C_2) = \sum_{i=2}^{8} P(i)$$ **Equation 21-36**

$$P(C_3) = \sum_{i=9}^{22} P(i)$$ **Equation 21-37**

$$P(C_4) = \sum_{i=23}^{169} P(i)$$ **Equation 21-38**

$$P(C_5) = \sum_{i=170}^{512} P(i)$$ **Equation 21-39**

The probability adjustment factors are then computed with Equation 21-40 through Equation 21-44.

$$AdjP(1) = \alpha[P(C_2) + 2P(C_3) + 3P(C_4) + 4P(C_5)]/1$$ **Equation 21-40**

$$AdjP(2) \text{ through } AdjP(8) = \alpha[P(C_3) + 2P(C_4) + 3P(C_5) - P(C_2)]/7$$ **Equation 21-41**

$$AdjP(9) \text{ through } AdjP(22) = \alpha[P(C_4) + 2P(C_5) - 3P(C_3)]/14$$ **Equation 21-42**

$$AdjP(23) \text{ through } AdjP(169) = \alpha[P(C_5) - 6P(C_4)]/147$$ **Equation 21-43**

$$AdjP(170) \text{ through } AdjP(512) = -\alpha[10P(C_5)]/343$$ **Equation 21-44**

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 21-45.

$$P'(i) = P(i) + AdjP(i)$$ **Equation 21-45**

5. 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.

Consequently, 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 the operating and geometric characteristics of the intersection in question. Although 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).

VEHICULAR VOLUMES

At intersections with relatively low vehicular 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 vehicular 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 above and depicted in Exhibit 21-2. 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 the time needed to cycle through all phases for the particular intersection, assuming random arrival of pedestrians. However, several other factors may 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.

6. BICYCLE MODE

The procedures described to estimate motor vehicle delay can be applied to bicycles that queue with motor vehicles on AWSC approaches. 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.

On an AWSC approach that 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 practice, 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; this results in lower effective bicycle delay compared with motor vehicle delay.

7. APPLICATIONS

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary engineering analysis.

Operational Analysis

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including analysis-hour demand volumes for each turning movement (vehicles per hour), 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, control delay, and queuing. The steps of the methodology, described in Section 3, are followed directly without modification.

The operational analysis methodology for AWSC intersections can also be used for design analysis and planning and preliminary engineering analysis.

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 or other desired performance measures.

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.

EXAMPLE PROBLEMS

Section 5 of Chapter 32, STOP-Controlled Intersections: Supplemental, provides two example problems that illustrate each of the computational steps involved in applying the motor vehicle method:

1. Analyze a single-lane, three-leg, AWSC intersection; and
2. Analyze a multilane, four-leg, AWSC intersection.

EXAMPLE RESULTS

The computations discussed in this chapter result in the estimation of control delay and LOS for each lane, for each approach, and for the entire intersection. When capacities are calculated with the iterative method described in this chapter, they also produce a volume-to-capacity ratio for each lane. This section provides some useful interpretations of these performance measures.

Level of Service

In general, LOS indicates 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.

As with other intersection types and controls, intersection LOS must be interpreted with caution. It can suggest acceptable operation of the intersection when, in reality, certain lanes or approaches (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 lanes or approaches. The analyst should verify that each lane or approach is providing acceptable operation and consider reporting the LOS for the poorest-performing lane or approach as a means of providing context to the interpretation of intersection LOS.

Volume-to-Capacity Ratio

The interpretation of volume-to-capacity ratios for AWSC intersections requires care due to the interdependence of the movements at the intersection. As discussed in the calculation of capacity in Step 12 of the methodology, the capacity of a subject approach is dependent on the performance of adjacent and opposing approaches, each of which depend on each other and the subject approach. As a result, unlike other procedures in which capacity is estimated directly, for AWSC intersections, capacity is estimated indirectly by holding the adjacent and opposing flows constant and loading the subject approach to the point of failure (a degree of utilization of 1.0). In addition, the degree of utilization, x, is used in the delay or queue equations rather than the capacity.

In general, a volume-to-capacity ratio greater than 1.0 is an indication of actual or potential breakdown. Assuming turning movement volumes are fixed, improvements that might be considered include the following:

- Basic changes in geometry (i.e., change in the number or use of lanes). The addition of lanes to an AWSC intersection approach typically changes the geometric groups for all movements with a resulting increase in departure headways, so a change to the subject approach to improve its performance may reduce the performance of other approaches; and
- Conversion to another type of intersection or control, or both (e.g., signal control or a roundabout).

Local guidelines should be consulted before potential improvements are developed.

8. REFERENCES

Some of these references are available in the Technical Reference Library in Volume 4.

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, Volume 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. 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.
4. 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.

CHAPTER 22
ROUNDABOUTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Roundabouts are intersections with a generally circular shape, characterized by yield on entry and circulation (counterclockwise in the United States) around a central island. Roundabouts have been used successfully throughout the world and are being used increasingly in the United States, especially since 1990.

This chapter presents concepts and procedures for analyzing these intersections. A Federal Highway Administration–sponsored project (*1*) has provided a comprehensive database of roundabout operations for U.S. conditions that is based on a study of 24 approaches at single-lane roundabouts and 37 approaches at multilane roundabouts. This study updates work conducted for National Cooperative Highway Research Program (NCHRP) Project 03-65 (*2*). The procedures that follow are based on these studies' 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.

CHAPTER ORGANIZATION

This chapter is organized into the following sections:

- Section 1 (this section) introduces the chapter.
- Section 2 presents basic concepts of the roundabout methodology, including capacity concepts and level-of-service (LOS) criteria.
- Section 3 describes the methodological details of the procedure, which include a step-by-step description of the analysis steps, a discussion of limitations of the method, and required data.
- Section 4 addresses extensions of the motorized vehicle analysis methodology specifically related to calibration of the model.
- Section 5 and Section 6, respectively, present pedestrian and bicycle evaluation considerations for roundabouts.
- Section 7 describes types of analysis, example problems included in Volume 4, and example results.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 23, Ramp Terminals and Alternative Intersections, discusses the analysis of interchange ramp terminals that are roundabouts.
- Chapter 33, Roundabouts: Supplemental, provides example problems of the roundabout methodology and additional methodological details, including model calibration.
- Section N, Roundabouts, in Part 2 of the *Planning and Preliminary Engineering Applications Guide to the HCM*, describes how to incorporate this chapter's methods and performance measures into a planning effort.

2. CONCEPTS

This chapter presents procedures for analyzing roundabouts, introduces the unique characteristics of roundabout capacity, and presents terminology specific to roundabouts.

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 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 types of models are applicable to roundabouts. Gap acceptance models, an example of an analytical model, 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 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. The procedure presented in this chapter, which is based on a recent analysis of U.S. field data, incorporates a combination of simple lane-based regression and gap acceptance models for both single-lane and double-lane roundabouts. As a result, the capacity models in this chapter focus on one entry of a roundabout at a time. The roundabout is considered in its entirety only in the determination of conflicting flow for the entry under consideration.

CAPACITY CONCEPTS

The capacity of a roundabout approach is directly influenced by flow patterns. The three flows of interest, the entering flow v_e, the circulating flow v_c, and the exiting flow v_{ex}, are shown in Exhibit 22-1.

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. The circulating flow directly conflicts with the entry flow, but 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 this effect may have an influence in some cases, including it did not significantly improve the overall fit of the capacity models to the data (*1*), and consequently it is not included in this chapter's models.

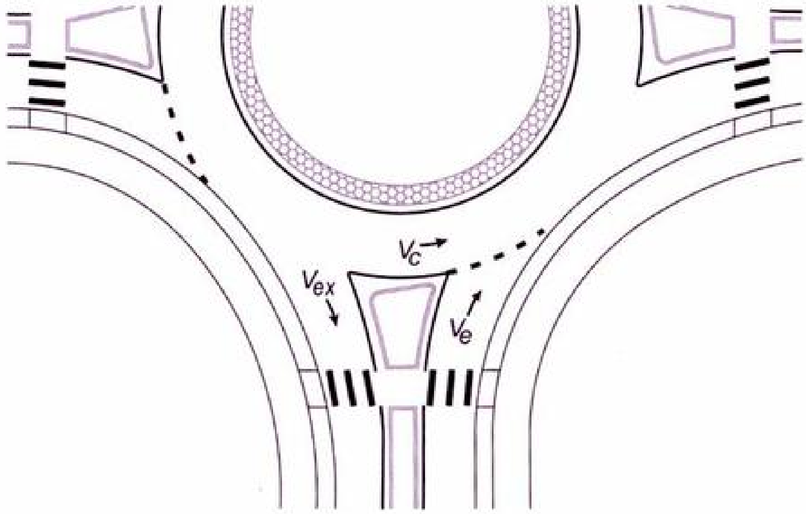


Exhibit 22-1
Analysis on One Roundabout Leg

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 (*3*, *4*). This effect was not identified in a more recent study (*2*) and has not been incorporated into this chapter's models.

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.

Both roundabout design practices and the public's use of roundabouts continue to mature in the United States. Research at the time of writing found variation in capacities throughout the United States. Such variation contributes to considerable spread in the data; more detail can be found in Chapter 33, Roundabouts: Supplemental. A likely source for this variation is differences in driver behavior in various regions, which may be influenced by the local driving culture and the density of roundabouts in an area. Other sources for the variation may include geometric features. Research in the United States was not able to isolate specific geometric factors relative to variations in driver behavior (*1*), although some international research has identified specific geometric contributions (*5*, *6*). Factors that may explain differences observed in the United States compared with other countries include limited use of turn indicators at roundabout exits, differences in vehicle types, and the effect the comparatively low use of YIELD-controlled intersections has on driver behavior.

U.S. research at the time of writing has not found significant increases in capacity over time, but some geographic areas showed significantly higher capacities than the national average.

Data collection in 2010 as part of national research resulted in a much larger and more saturated data set that exhibited higher capacities than previously reported in the United States. The capacities presented here are believed to be higher primarily due to the larger and more saturated data set and not primarily due to an increase in capacity over time. Although it has generally been assumed roundabout capacity values in the United States will increase as drivers become more experienced with roundabouts, it has not been possible to provide direct evidence of this characteristic from the available data. Data examined at two roundabouts observed under saturated conditions in 2003 and 2012 revealed no significant change in observed capacities. However, Carmel, Indiana, a city with a large number of roundabouts, had significantly higher roundabout capacity values than average for U.S. conditions (*1*).

Single-Lane Roundabouts

The capacity of a single entry lane conflicted by one circulating lane (e.g., a single-lane roundabout, illustrated in Exhibit 22-2) is based on the conflicting flow. The equation for estimating the capacity is given as Equation 22-1.

Equation 22-1

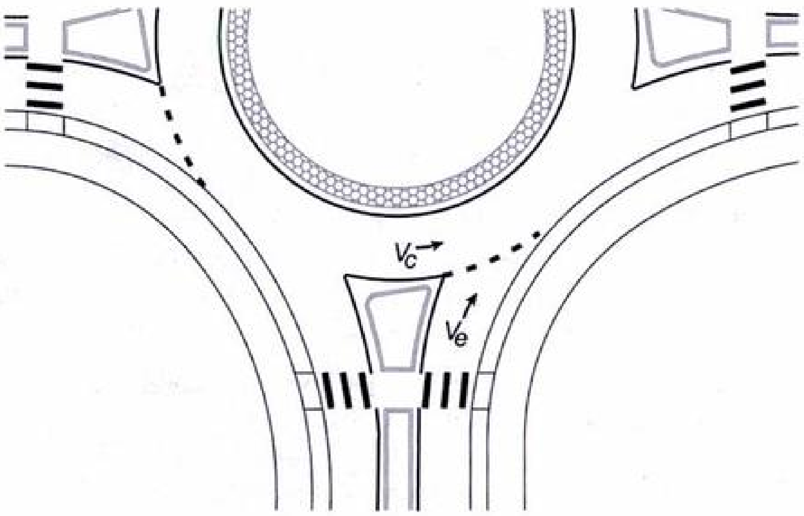
$$c_{e,pce} = 1{,}380e^{(-1.02\times10^{-3})v_{c,pce}}$$

where

$c_{e,pce}$ = lane capacity, adjusted for heavy vehicles (pc/h); and

$v_{c,pce}$ = conflicting flow rate (pc/h).

Exhibit 22-2
Example of One-Lane Entry Conflicted by One Circulating Lane



The capacity model given above reflects observations made at U.S. roundabouts in 2012 (*1*). Considerable variation in capacity was observed in various regions of the country and with different sites within a region. Therefore, local calibration of the capacity models is recommended to best reflect local driver behavior. This topic is discussed later in this chapter and in Chapter 33.

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 for multilane roundabouts than single-lane roundabouts.

The definition of headways and gaps for multilane facilities is also 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 drivers in the right entry lane, in particular, is imperfect and difficult to quantify with a simple gap acceptance model. This difficulty 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 (*7*).

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 below reflects observations made at U.S. roundabouts in 2012 (*1*). As 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

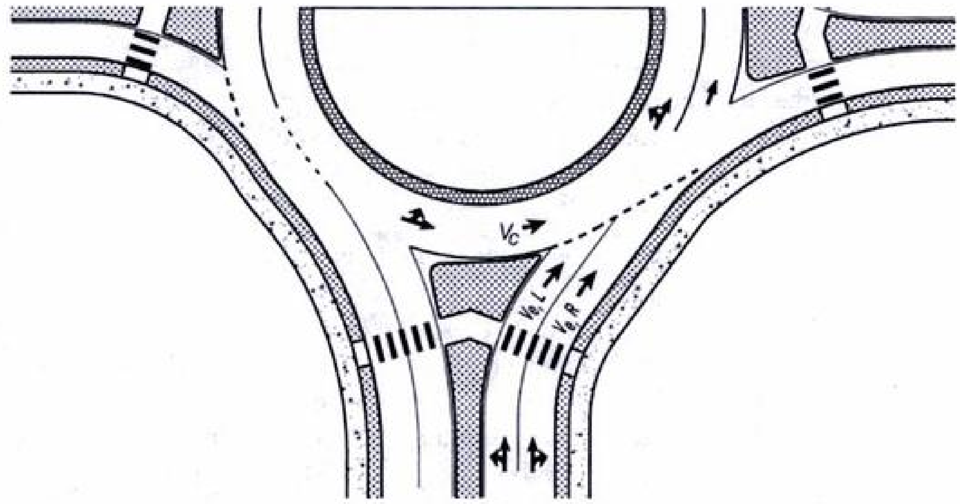
Equation 22-2 gives the capacity of each entry lane conflicted by one circulating lane (illustrated in Exhibit 22-3).

Equation 22-2

$$c_{e,pce} = 1{,}420e^{(-0.91\times10^{-3})v_{c,pce}}$$

where all variables are as defined previously.

Exhibit 22-3
Example of Two-Lane Entry Conflicted by One Circulating Lane



Capacity for One-Lane Entries Conflicted by Two Circulating Lanes

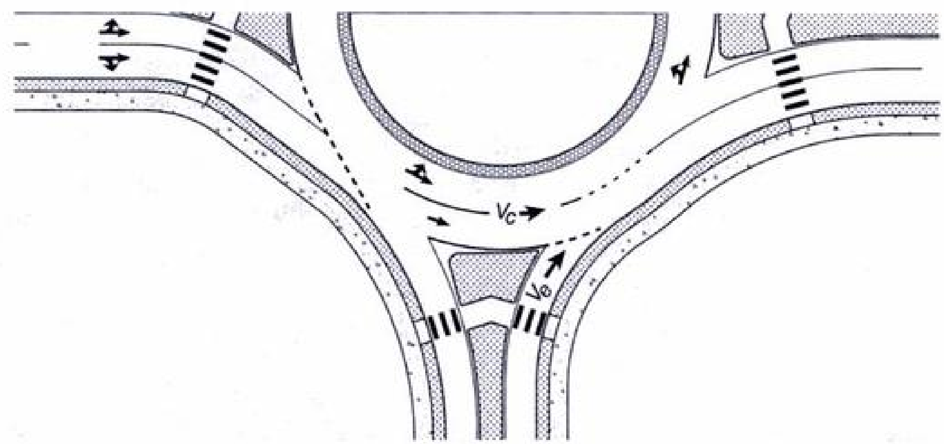
Equation 22-3 gives the capacity of a one-lane roundabout entry conflicted by two circulating lanes (illustrated in Exhibit 22-4).

Equation 22-3

$$c_{e,pce} = 1{,}420e^{(-0.85\times10^{-3})v_{c,pce}}$$

where all variables are as defined previously ($v_{c,pce}$ is the total of both lanes).

Exhibit 22-4
Example of One-Lane Entry Conflicted by Two Circulating Lanes



Capacity for Two-Lane Entries Conflicted by Two Circulating Lanes

Equation 22-4 and Equation 22-5 give the capacity of the right and left lanes, respectively, of a two-lane roundabout entry conflicted by two circulating lanes (illustrated in Exhibit 22-5).

Equation 22-4

$$c_{e,R,pce} = 1{,}420e^{(-0.85\times10^{-3})v_{c,pce}}$$

Equation 22-5

$$c_{e,L,pce} = 1{,}350e^{(-0.92\times10^{-3})v_{c,pce}}$$

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).

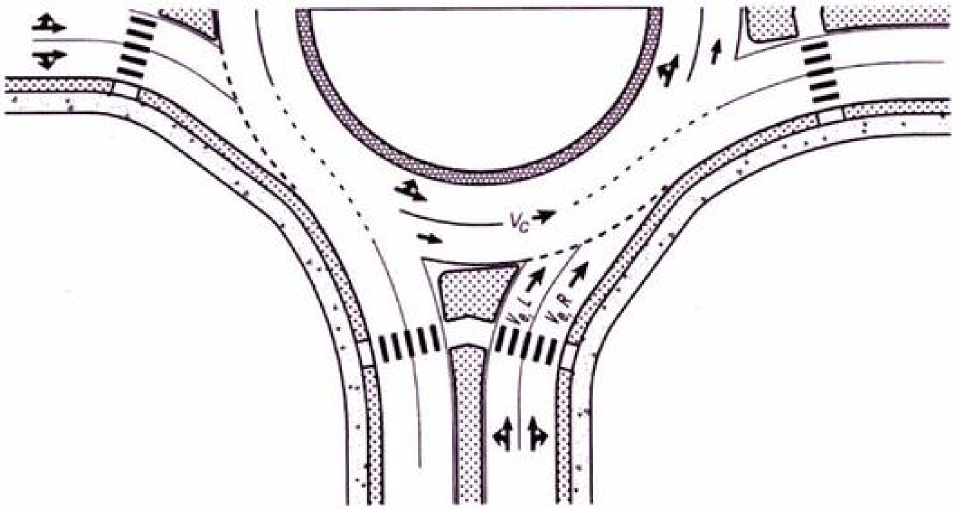


Exhibit 22-5
Example of Two-Lane Entry Conflicted by Two Circulating Lanes

The calculated capacities for each lane in passenger car equivalents per hour are adjusted back to vehicles per hour, as described later in this section.

Exhibit 22-6 presents a plot showing the entry capacity equations (Equation 22-1 through Equation 22-5).

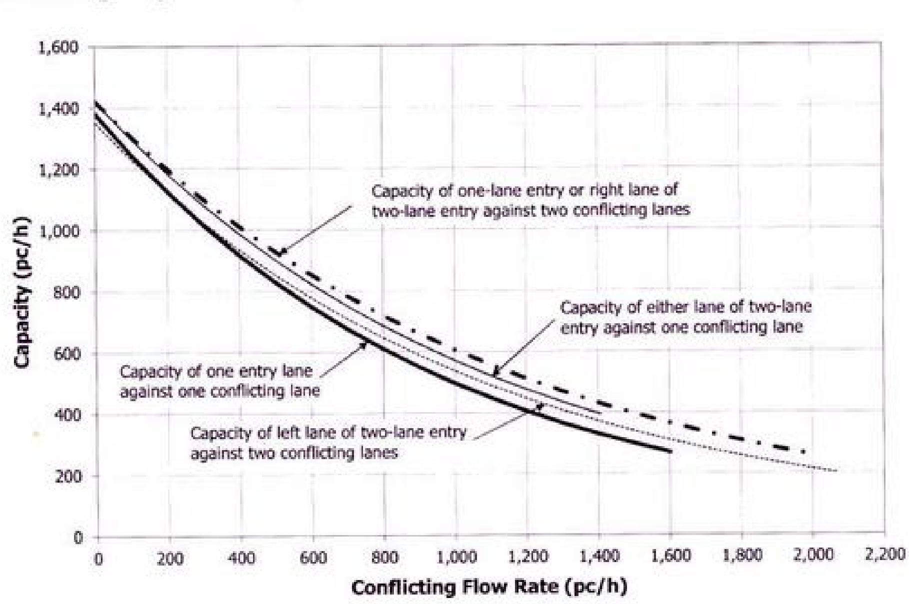


Exhibit 22-6
Capacity of Single-Lane and Multilane Entries

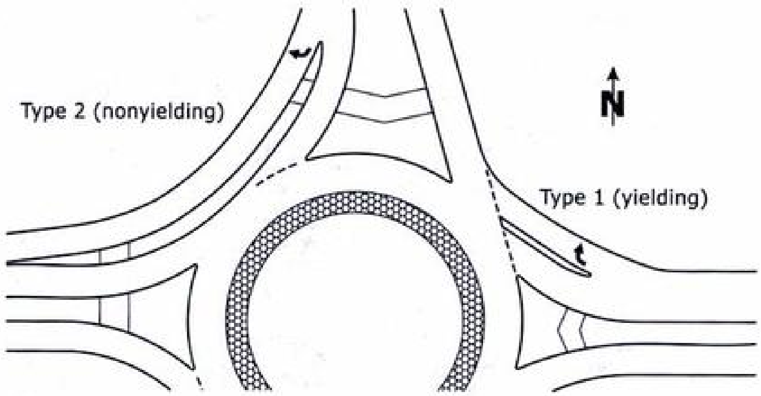
Right-Turn Bypass Lanes

Two common types of right-turn bypass lanes are used at both single-lane and multilane roundabouts. These are illustrated in Exhibit 22-7.

The following sections describe each type of bypass lane. In the United States, drivers in both types of bypass lanes 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.

The bypass lane capacity procedure does not include the effect of drivers yielding to pedestrians.

Exhibit 22-7
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 evaluated in the most recent national research (*1*). 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 22-6.

Equation 22-6

$$c_{\text{bypass},pce} = 1{,}380e^{(-1.02\times10^{-3})v_{ex,pce}}$$

The capacity for a bypass lane opposed by two exiting lanes can be approximated by using Equation 22-7.

Equation 22-7

$$c_{\text{bypass},pce} = 1{,}420e^{(-0.85\times10^{-3})v_{ex,pce}}$$

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 (*8*) suggests 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. 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 to provide basic lane continuity along a corridor, regardless of the exit volume. Further guidance can be found in an NCHRP report (*7*).

LEVEL-OF-SERVICE CRITERIA

LOS criteria for motorized vehicles in roundabouts are given in Exhibit 22-8. 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 22-8 are based on the 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 the similar control delay formulation.

Exhibit 22-8
LOS Criteria: Motorized Vehicle 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.

3. MOTORIZED VEHICLE CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

This section focuses on the operation of roundabouts. This version of the roundabout analysis procedures is primarily based on studies conducted for the Federal Highway Administration (*1*) updating work conducted for NCHRP Project 03-65 (*2*).

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 differ significantly from that experienced at roundabouts and thus cannot be accurately modeled by using the procedures in this section. More detail on the differentiation between roundabouts and other circular intersections can be found elsewhere (*7*).

Spatial and Temporal Limits

The analytical procedure presented in this section assumes the analysis boundaries are the roundabout itself, including associated pedestrian crosswalks. Alternative tools discussed in this section can, in some cases, expand the analysis boundaries to include adjacent intersections. The methodology presented here discusses motorized vehicles, pedestrians, and bicycles.

The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This method produces the following performance measures:

- Volume-to-capacity ratio,
- Control delay,
- LOS based on control delay, and
- 95th percentile queue length.

Limitations of the Methodology

The procedures presented in this section cover many of the typical situations a user may encounter in practice. However, for some applications alternative tools can produce a more accurate analysis. The following limitations, stated earlier in this section, may be addressed by using available simulation tools. The conditions beyond the scope of this chapter that are treated explicitly by alternative tools include

- Pedestrian signals or hybrid beacons at roundabout crosswalks,
- Metering signals on one or more approaches,
- Adjacent signals or roundabouts,
- Priority reversal under extremely high flows,

Priority reversal can occur when entering traffic dominates an entry, causing circulating traffic to yield.

- High pedestrian or bicycle activity levels,
- More than two entry lanes on an approach, or
- Flared entry lanes.

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.

A few of the more common applications of alternative tools to overcome the limitations of the procedures presented in this section are discussed in the following subsections.

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* (*9*), 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 motorized vehicle capacity; and
    - Crossing situations in which pedestrians with vision or other disabilities may not receive equivalent access to the crossing; such access is a legal requirement in the United States under the Americans with Disabilities Act and is regulated by the U.S. Access Board (*10*).
- *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.

Although 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 (*5*).

The methodology presented in this section 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 section 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, 2*) for this chapter's methodology, so the analyst should use care in estimating calibration parameters.

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 (*7*).

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., *5, 6, 11, 12*). 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 they 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., *13, 14*). 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 section are based on extensive research supported by a significant quantity of field data. They 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, but simulation-based tools 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 reviews.

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 adjustment factors 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 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.

REQUIRED INPUT DATA AND SOURCES

Exhibit 22-9 lists the information necessary to apply the motorized vehicle methodology and suggests potential sources for obtaining these data. It also suggests default values for use when intersection-specific information is not available.

No default values have been developed specifically for roundabouts. However, a comprehensive presentation of potential default values for interrupted-flow facilities is available (*15*) with specific recommendations summarized in its Chapter 3, Recommended Default Values. These defaults cover the key characteristics of peak hour factor and percentage of heavy vehicles. Recommendations are based on 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.

Default values for lane utilization on two-lane roundabout approaches are not provided in the above reference (*15*). In these cases, in the absence of field data, the effect of lane utilization imbalance can be approximated by using the suggested default values given in Exhibit 22-9.

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.

Exhibit 22-9
Required Input Data, Potential Data Sources, and Default Values for Roundabout Motorized Vehicle Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Number and configuration of lanes on each approach	Design plans, road inventory	Must be provided
	Demand Data	
Hourly turning movement demand volume (veh/h) AND peak hour factor OR Hourly turning movement demand flow rate (veh/h)	Field data, modeling	Must be provided
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor (decimal)	Field data	0.92
Heavy-vehicle percentage (%)	Field data	3%
Lane utilization	Field data	Left–through + through–right: % traffic in left lane: 0.47 [a] % traffic in right lane: 0.53 [a] Left–through–right + right: % traffic in left lane: 0.47 [a] % traffic in right lane: 0.53 [a] Left + left–through–right: % traffic in left lane: 0.53 [a] % traffic in right lane: 0.47 [a]

Note: [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.

COMPUTATIONAL STEPS

The capacity of a given approach is computed by using the process illustrated in Exhibit 22-10.

Exhibit 22-10
Roundabout Methodology

Step 1: Convert movement demand volumes to flow rates
↓
Step 2: Adjust flow rates for heavy vehicles
↓
Step 3: Determine circulating and exiting flow rates
↓
Step 4: Determine entry flow rates by lane
↓
Step 5: Determine the capacity of each entry lane and bypass lane as appropriate in passenger car equivalents
↓
Step 6: Determine pedestrian impedance to vehicles
↓
Step 7: Convert lane flow rates and capacities into vehicles per hour
↓
Step 8: Compute the volume-to-capacity ratio for each lane
↓
Step 9: Compute the average control delay for each lane
↓
Step 10: Determine LOS for each lane on each approach
↓
Step 11: Compute the average control delay and determine LOS for each approach and the roundabout as a whole
↓
Step 12: Compute 95th percentile queues for each lane

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 four.

For analysis of projected conditions or when 15-min data are not available, hourly demand volumes for each movement are converted to peak 15-min demand flow rates in vehicles per hour, as shown in Equation 22-8, through the use of a peak hour factor for the intersection.

Equation 22-8

$$v_i = \frac{V_i}{PHF}$$

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.

If PHF is used, a single intersectionwide PHF should be used rather than movement-specific or approach-specific PHFs. If individual approaches or movements peak at different times, a series of 15-min analysis periods that encompasses the peaking should be considered.

The use of a peak 15-min traffic count multiplied by four is preferred for existing conditions when traffic counts are available. The use of a 1-h demand volume divided by a peak hour factor is preferred with projected volumes or with projected volumes that have been added to current volumes.

If peak hour factors are used, a single peak hour factor for the entire intersection is generally preferred to decrease the likelihood of creating demand scenarios with conflicting volumes that are disproportionate to the actual volumes during the 15-min analysis period. If peak hour factors for each individual approach or movement are used, they are likely to generate demand volumes from one 15-min period that are in apparent conflict with demand volumes from another 15-min period, but in reality these peak volumes do not occur at the same time. Furthermore, to determine individual approach or movement peak hour factors, actual 15-min count data are likely available, permitting the determination of actual 15-min demand and avoiding the need to use a peak hour factor. In the event individual approaches or movements are known to have substantially different peaking characteristics or peak during different 15-min periods within the hour, a series of 15-min analysis periods that encompasses the peaking should be considered instead of a single analysis period using a single peak hour factor for the intersection.

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 22-11.

Exhibit 22-11
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 22-9 and Equation 22-10.

Equation 22-9

$$v_{i,pce} = \frac{v_i}{f_{HV}}$$

Equation 22-10

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

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.

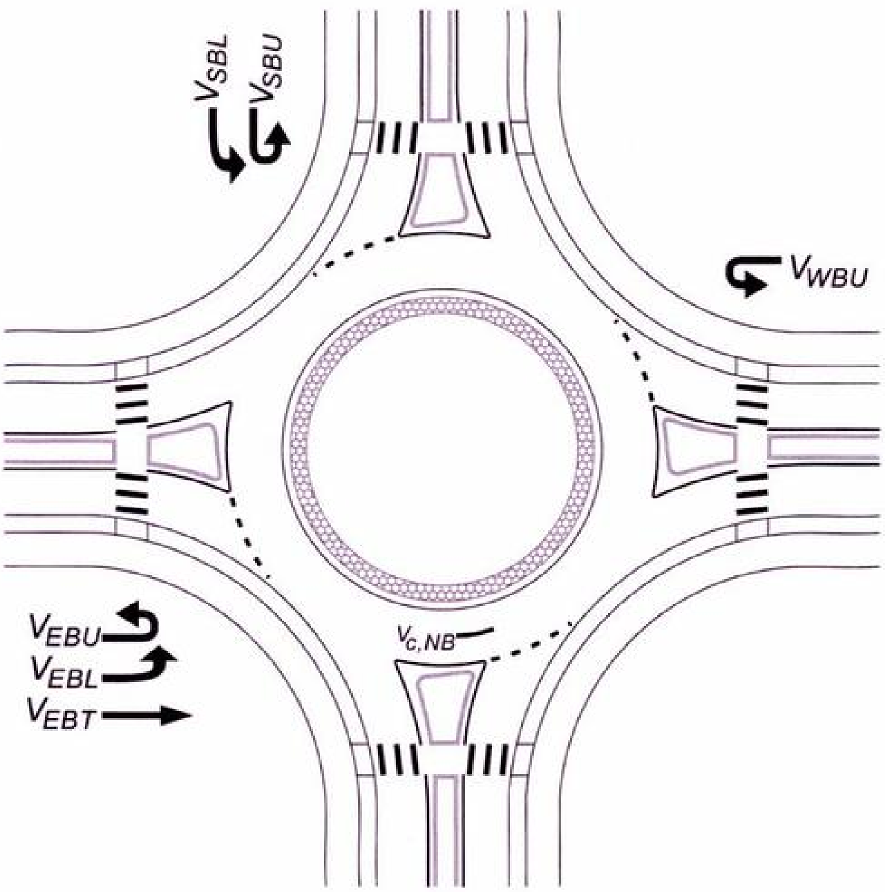
Step 3: Determine Circulating and Exiting Flow Rates

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 22-12 and numerically in Equation 22-11. All flows are in passenger car equivalents.

Exhibit 22-12
Calculation of Circulating Flow



Equation 22-11

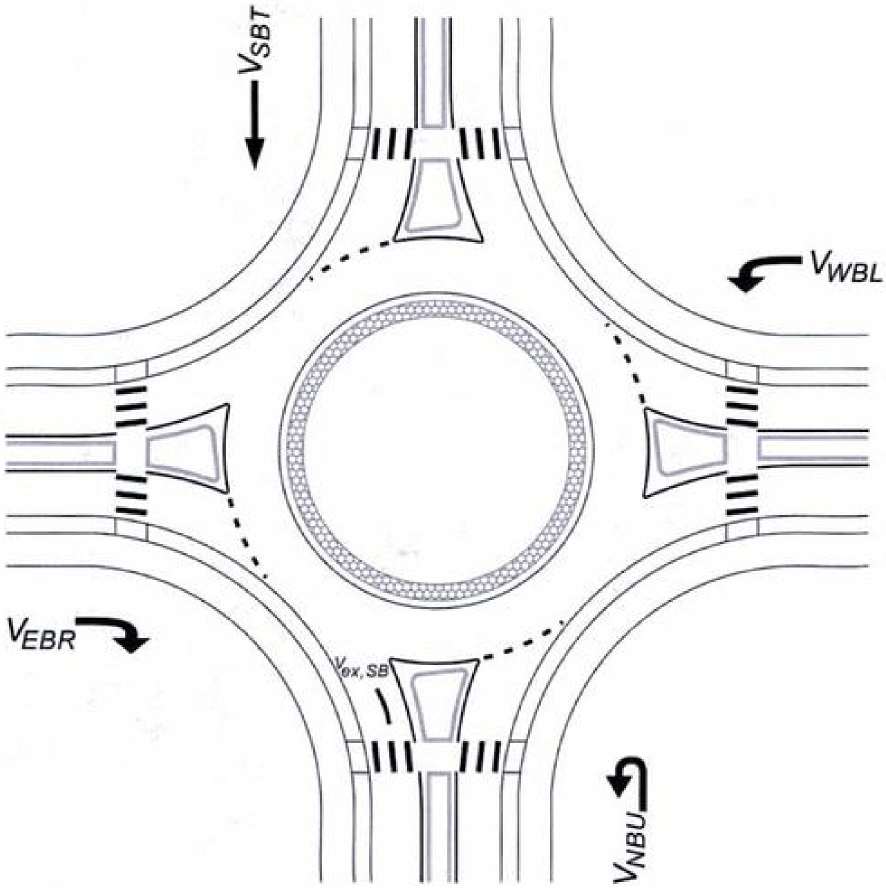
$$v_{c,NB,pce} = v_{WBU,pce} + v_{SBL,pce} + v_{SBU,pce} + v_{EBT,pce} + v_{EBL,pce} + v_{EBU,pce}$$

Exiting Flow Rate

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.

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 22-13 and numerically in Equation 22-12. 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.

Exhibit 22-13
Calculation of Exiting Flow



Equation 22-12

$$v_{ex,SB,pce} = v_{NBU,pce} + v_{WBL,pce} + v_{SBT,pce} + v_{EBR,pce} - v_{EBR,pce,bypass}$$

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 configuration may differ from the designated lane assignment based on the specific turning movement patterns being analyzed. These assumed lane assignments are given in Exhibit 22-14. For intersections with a different number of legs, the analyst should exercise reasonable judgment in assigning volumes to each lane.

Exhibit 22-14
Assumed (de facto) Lane Assignments

Designated Lane Assignment	Assumed Lane Assignment
Left–through, through–right	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
Left, left–through–right	If $v_T + v_{R,e} > v_U + v_L$: L, TR (de facto through–right lane) Else L, LTR
Left–through–right, right	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; 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 22-15.

Exhibit 22-15
Volume Assignments for Two-Lane Entries

Case	Assumed Lane Assignment	Left Lane	Right Lane
1	Left, through–right	$v_U + v_L$	$v_T + v_{R,e}$
2	Left–through, right	$v_U + v_L + v_T$	$v_{R,e}$
3	left–through, through–right	$(\%LL)v_e$	$(\%RL)v_e$
4	Left, left–through–right	$(\%LL)v_e$	$(\%RL)v_e$
5	Left–through–right, right	$(\%LL)v_e$	$(\%RL)v_e$

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; R = right; $\%RL$ = percentage of entry traffic using the right lane; $\%LL$ = percentage of entry traffic using the left lane. $\%LL + \%RL = 1$.

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 above. Capacity equations for entry lanes are summarized in Exhibit 22-16; capacity equations for bypass lanes are summarized in Exhibit 22-17.

Exhibit 22-16
Capacity Equations for Entry Lanes

Entering Lanes	Conflicting Circulating Lanes	Capacity Equation
1	1	Equation 22-1
2	1	Each lane: Equation 22-2
1	2	Equation 22-3
2	2	Right lane: Equation 22-4; left lane: Equation 22-5

Exhibit 22-17
Capacity Equations for Bypass Lanes

Conflicting Exiting Lanes	Capacity Equation
1	Equation 22-6
2	Equation 22-7

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 22-18 can be used to approximate this effect (*8*). These equations are illustrated in Exhibit 22-19 and are based on the assumption that pedestrians have absolute priority.

Exhibit 22-18
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	$f_{ped} = \dfrac{1{,}119.5 - 0.715 v_{c,pce} - 0.644 n_{ped} + 0.00073 v_{c,pce} n_{ped}}{1{,}068.6 - 0.654 v_{c,pce}}$

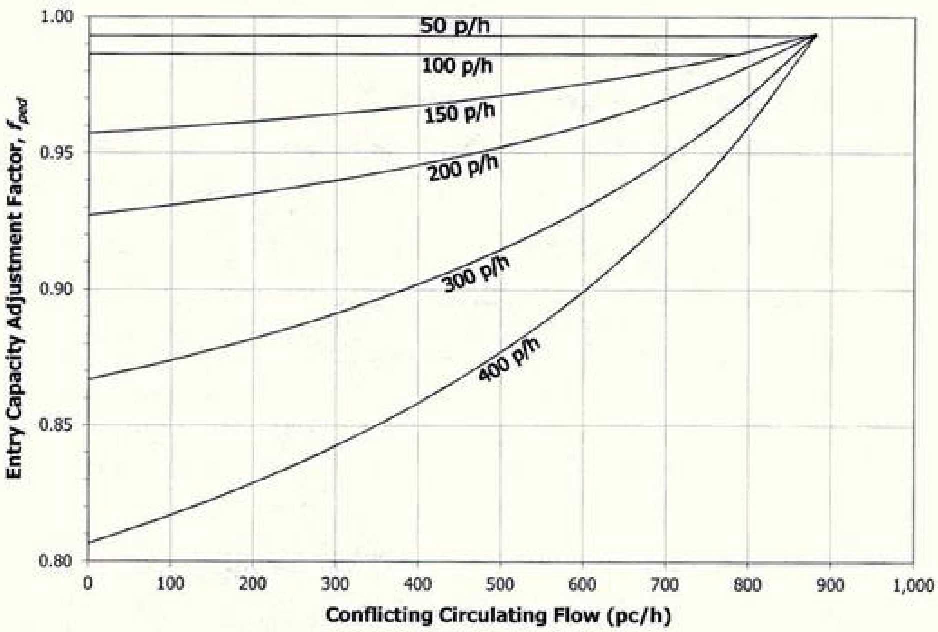
where

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 22-19
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 22-20 can be used to approximate the effect of pedestrians (*8*). These equations are illustrated in Exhibit 22-21 and share the assumption as before that pedestrians have absolute priority.

Exhibit 22-20
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	$f_{ped} = \min\left[\frac{1{,}260.6 - 0.329v_{c,pce} - 0.381n_{ped}}{1{,}380 - 0.5v_{c,pce}}, 1\right]$

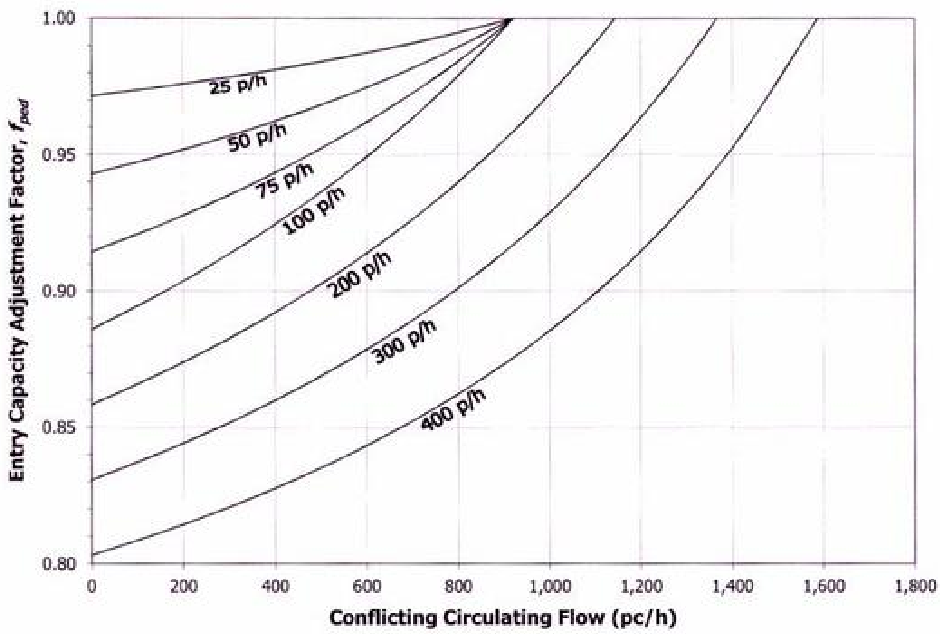
where

f_{ped} = entry capacity adjustment factor for pedestrians,

n_{ped} = number of conflicting pedestrians per hour (p/h), and

$v_{c,pce}$ = conflicting vehicular flow rate in the circulatory roadway (pc/h).

Exhibit 22-21
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 22-13.

Equation 22-13

$$v_i = v_{i,PCE} f_{HV,e}$$

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 22-14.

Equation 22-14

$$c_i = c_{i,PCE} f_{HV,e} f_{ped}$$

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 22-15.

Equation 22-15

$$f_{HV,e} = \frac{f_{HV,U}v_{U,PCE} + f_{HV,L}v_{L,PCE} + f_{HV,T}v_{T,PCE} + f_{HV,R,e}v_{R,e,PCE}}{v_{U,PCE} + v_{L,PCE} + v_{T,PCE} + v_{R,e,PCE}}$$

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 22-16. Both input values are in vehicles per hour.

Equation 22-16

$$x_i = \frac{v_i}{c_i}$$

where

x_i = volume-to-capacity ratio of subject lane i,

v_i = demand flow rate of subject lane i (veh/h), and

c_i = capacity of 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 control delays can be predicted in a manner generally similar to that used for other unsignalized intersections. Equation 22-17 shows the model that should be used to estimate average control delay for each lane of a roundabout approach.

Equation 22-17

The third term of this equation uses the calculated volume-to-capacity ratio or 1, whichever is less.

$$d = \frac{3{,}600}{c} + 900T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{450T}}\right] + 5 \times \min\,[x, 1]$$

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 22-17 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 model used above to estimate average control delay assumes 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 analysis period length. 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 analysis 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 (*16*).

Step 10: Determine LOS for Each Lane on Each Approach

LOS for each lane on each approach is determined by using Exhibit 22-8 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 22-18. The volume in the bypass lane should be included in the delay calculation for the approach. LOS for each approach is determined by using Exhibit 22-8 and the computed or measured values of control delay.

$$d_{\text{approach}} = \frac{d_{LL} v_{LL} + d_{RL} v_{RL} + d_{\text{bypass}} v_{\text{bypass}}}{v_{LL} + v_{RL} + v_{\text{bypass}}}$$

Equation 22-18

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, as shown in Equation 22-19. LOS for the intersection is determined by using Exhibit 22-8 and the computed or measured values of control delay.

Equation 22-19

$$d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}$$

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 22-20.

Equation 22-20

$$Q_{95} = 900T\left[x - 1 + \sqrt{(1-x)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{150T}}\right]\left(\frac{c}{3{,}600}\right)$$

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.

4. EXTENSION TO THE MOTORIZED VEHICLE METHODOLOGY

INTRODUCTION

As noted in Section 2, research has found variation in roundabout capacities throughout the United States; differences in driver behavior and geometric factors are potential causes of this variation (*1*). To address this potential for variation, this section presents a method for calibrating the HCM capacity models for local conditions.

CALIBRATION OF CAPACITY MODELS

The capacity models presented in Section 3 can be generalized as the Siegloch model (*17*) by using the expressions in Equation 22-21 through Equation 22-23.

$$c_{pce} = Ae^{(-Bv_c)}$$

Equation 22-21

$$A = \frac{3{,}600}{t_f}$$

Equation 22-22

$$B = \frac{t_c - (t_f/2)}{3{,}600}$$

Equation 22-23

Field measures of critical headway and follow-up headway can be used to calibrate the capacity models.

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).

With this formulation, the capacity model can be calibrated by using two parameters: the critical headway t_c and the follow-up headway t_f.

Research (*1*) has found that a reasonable calibration can be made by using only field measurements of follow-up headway to calculate the intercept *A* and retaining the value for *B*. This procedure recognizes the difficulty in measuring critical headway in the field.

Examples illustrating these calibration procedures are provided in Chapter 33, Roundabouts: Supplemental.

5. PEDESTRIAN MODE

Limited research has been performed 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 type of pedestrian right-of-way is somewhat different from those in 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 on pedestrians in the United States has focused on 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 (*10*). Various treatments have been or are 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 20). However, in many cases, alternative tools will produce more accurate results. These are discussed in Section 7, Applications.

Techniques to analyze the operational performance of pedestrians as provided in Chapter 20, 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 (*18*). This variation makes modeling of pedestrian interactions imprecise. As a result, models to analyze vehicular effects on pedestrian travel should be applied with caution.

6. 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 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, a bicyclist may either circulate as a motorized vehicle or as a pedestrian. If bicyclists are circulating as motorized vehicles, their effect can be approximated by combining bicyclist flow rates with other vehicles by using a passenger-car-equivalent factor of 0.5 (*7*). 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 (*7*).

Use a passenger-car-equivalent factor of 0.5 for bicycles when treating them as motorized vehicles.

7. APPLICATIONS

TYPES OF ANALYSIS

This chapter's methodology 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

Design analysis determines the geometric configuration of a roundabout to produce a desired operational performance.

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.

Planning and Preliminary Engineering Analysis

Planning and preliminary engineering analysis is used to evaluate future conditions for which assumptions and estimates must be made.

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 percentage of heavy vehicles and peak hour factor are typically estimated (or defaults are used) when planning applications are performed.

EXAMPLE PROBLEMS

Section 3 of Chapter 33, Roundabouts: Supplemental, provides two example problems that go through each of the computational steps involved in applying the motorized vehicle method:

1. Analyze a single-lane roundabout with bypass lanes, and
2. Analyze a multilane roundabout.

EXAMPLE RESULTS

Analysis of roundabouts is commonly performed as part of an alternatives analysis with STOP-controlled or signalized alternatives to determine the most appropriate intersection form and control. 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* (9).

In evaluating the overall performance of roundabouts, it is important to consider measures of effectiveness in addition to control delay, such as volume-to-capacity ratios for individual lanes, average queue lengths, and 95th percentile queue lengths. By focusing on a single measure of effectiveness for the worst lane only, users may make less effective traffic control decisions. The analyst using HCM or other operational analysis methods should carefully balance the operational effect for motor vehicles of a change in lane configuration computed with other performance measures that may be important in the overall evaluation; these may include the operational performance of other modes, safety performance, impacts to the built and natural environment, and life-cycle costs.

An example of this consideration occurs in determining the appropriate lane configuration for a roundabout. Consider as an example a roundabout at the intersection of a four-lane major street (such as a state highway) with a two-lane minor street (such as a local street). At a roundabout, each minor-street entry would typically have a single lane opposed by two conflicting circulating lanes. Under typical traffic flows for a configuration like this, the major-street traffic may be much heavier than the traffic on the minor street, resulting in low entry flows and high circulating flows for each of the minor-street entries. As the right side of Exhibit 22-22 shows, the high conflicting flows result in a low entry capacity. Consequently, a small change in entering flows can result in a large change in the volume-to-capacity ratio for the entry, as well as corresponding increases in control delay and queues. Increasing the entry from one to two lanes can reduce the volume-to-capacity ratio and associated control delays and queues for the motor vehicle mode for this minor-street entry. However, this lane increase comes at a potential cost to other performance measures, including safety performance for all modes, control delay for pedestrians, accessibility for pedestrians, and other costs and impacts. The analyst should carefully balance these trade-offs to determine the appropriate course of action and not rely exclusively on the operational performance of motor vehicles to make this decision.

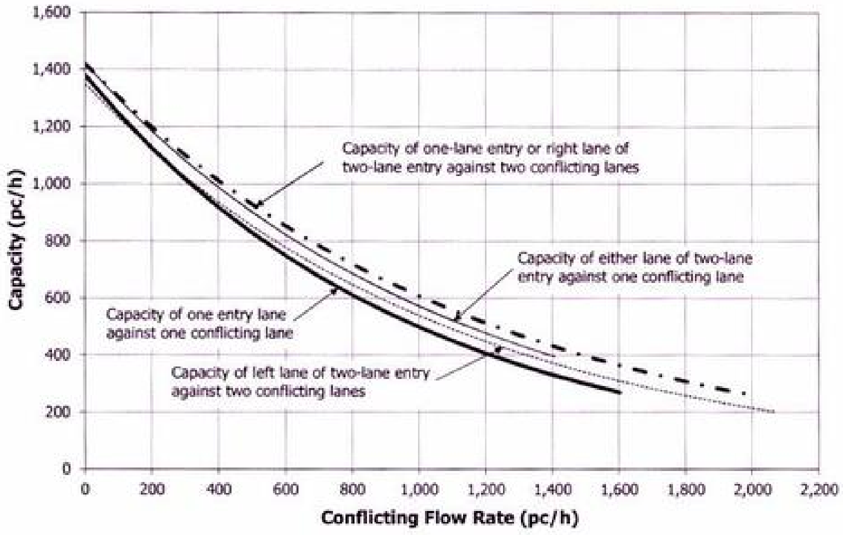


Exhibit 22-22
Illustration of Capacity for Low Entry Flows and High Circulating Flows

8. REFERENCES

Some of these references are available in the Technical Reference Library in Volume 4.

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CHAPTER 23
RAMP TERMINALS AND ALTERNATIVE INTERSECTIONS

CONTENTS

LIST OF EXHIBITS

Part A: Distributed Intersection Concepts

1. INTRODUCTION

OVERVIEW

This chapter presents a methodology for the analysis of interchange ramp terminals, alternative intersections, and alternative interchanges. The interchange ramp terminal methodology was developed primarily on the basis of research conducted through the National Cooperative Highway Research Program (*1–3*) and elsewhere (*4*). The alternative intersection and interchange methodology was based on research conducted through the Federal Highway Administration (FHWA) (*5*).

Interchange ramp terminals are critical components of the highway network. They provide the connection between various highway facilities (freeway–arterial, arterial–arterial, etc.), and thus their efficient operation is essential. Interchanges are typically 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.

Alternative intersections are created by rerouting one or more movements from their usual places to secondary junctions. Often, the rerouted movements are left turns. Alternative intersection and interchange designs have significantly reduced travel times and delays in many areas, compared with conventional intersection designs. Some designs have substantially reduced the number of conflict points between vehicles and thus increased overall safety. In addition, the alternative designs can often be implemented with minimal disruptions to existing right-of-way. Given the relatively low cost of implementation for many of these designs, the combination of improved mobility and safety has produced outstanding benefit–cost ratios within economic analyses. By relocating or eliminating problematic movements, the alternative designs can efficiently mitigate congestion at surface street–freeway interchanges and at signalized intersections.

Both interchange ramp terminals and alternative at-grade intersections are discussed in this chapter, because they combine multiple intersections in a cluster. "Distributed intersections" consist of groups of two or more intersections that, by virtue of close spacing and displaced or distributed traffic movements, are operationally interdependent and are thus best analyzed as a single unit. The most common distributed intersections are interchange ramp terminals, but other alternative intersection forms—such as those involving displaced left-turn movements—also fall into this category.

"Distributed intersections" consist of groups of two or more intersections that, by virtue of close spacing and displaced or distributed traffic movements, are operationally interdependent and, thus, best analyzed as a single unit.

Research has not yet been performed on pedestrian and bicyclist perceptions of service quality specific to interchange ramp terminals and alternative intersections. Therefore, no service measures are provided in this chapter for those modes. Several FHWA publications (e.g., *6–8*) discuss designing for non-automobile users of alternative intersections and interchanges.

CHAPTER ORGANIZATION

Part A of Chapter 23 provides an overview of alternative intersection and interchange concepts. Within this part, Section 2 documents and describes a number of common concepts associated with interchanges and alternative intersections. This section lists the unique elements and summarizes the shared attributes of such facilities. It further discusses the need for translating between *turning movement volume demands* at each intersection approach and *origin–destination demands* across the entire intersection or interchange. The section discusses issues related to distributed intersections and interchanges, including an origin–destination framework. To facilitate unbiased comparisons among distributed intersection types, this section introduces a discussion of *experienced travel time* and delay—consisting of diverted path delay and control delay.

Part B of Chapter 23 focuses on the evaluation of surface street–freeway interchanges. Following the Section 1 overview, Section 2 describes the features of diamond interchanges, partial cloverleafs, single-point urban interchanges, diverging diamond interchanges, roundabout interchanges, and others. Section 3 discusses the core evaluation methodology, including scope, required data, and computational steps. Section 4 describes methodology extensions for interchanges with roundabouts and interchanges with STOP and YIELD signs, and it describes a specific procedure for interchange type selection. Section 5 presents applications of the Part B methodology, including example results, analysis types, and the pros and cons of analyzing surface street–freeway interchanges with alternative tools.

Part C of Chapter 23 focuses on the evaluation of alternative intersections. After the Section 1 overview, Section 2 describes the features of restricted crossing U-turn intersections (also known as superstreets), median U-turn intersections, displaced left-turn intersections (also known as continuous flow intersections), and others. Section 3 discusses the core evaluation methodology, including scope, required data, and computational steps. Section 4 describes methodology extensions for alternative intersection designs not covered in Section 3. Section 5 presents applications of the Part C methodology, including example results, analysis types, and the pros and cons of analyzing alternative intersections with alternative tools.

RELATED HCM CONTENT

Other *Highway Capacity Manual* (HCM) content related to this chapter includes the following:

- Chapter 4, Traffic Operations and Capacity Concepts, explains some of the fundamental concepts behind capacity analysis of interrupted-flow facilities, which encompass all intersections and interchanges. The chapter discusses capacity, volume, headway, stops, queuing, density, flow rate, lost time, control delay, saturation flow, peak hour factors, and different types of speeds.
- Chapter 5, Quality and Level-of-Service Concepts, discusses methods for assessing quality of service and level of service (LOS) for all surface and freeway facilities.

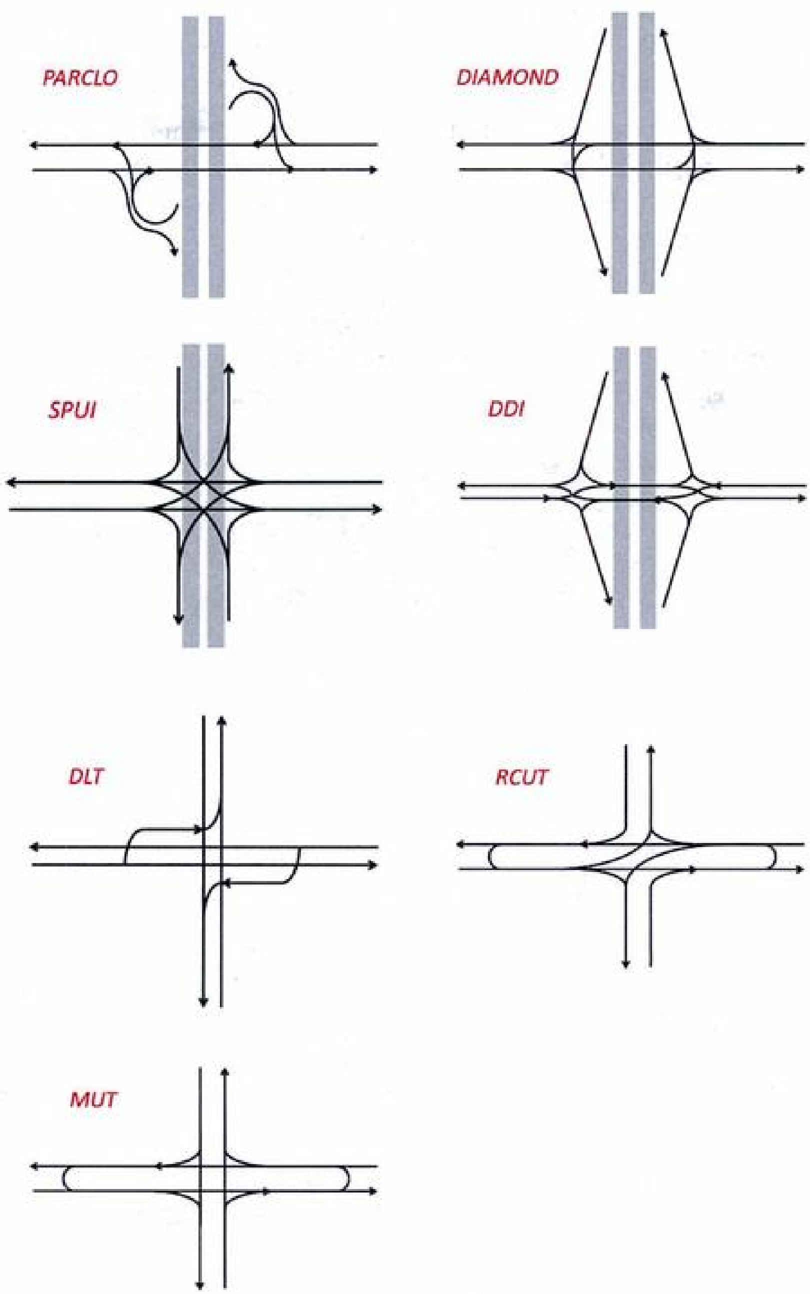
- Chapter 19, Signalized Intersections, contains detailed procedures and discussions for signalized intersection analysis. Many of the concepts introduced in this chapter establish the baseline for analysis of complex intersections and interchanges. These concepts include lane group determination, signal timing determination, saturation flow rate adjustment, lane utilization, and control delay estimation.
- Chapter 18, Urban Street Segments, extends the Chapter 19 signalized intersection procedures and discussions to account for adjacent intersection effects. Some of these concepts are also prerequisites for the analysis of complex intersections and interchanges. These concepts include flow profile determination, coordinated signal timing, travel time estimation, and estimation of the proportion of vehicles arriving on green.
- Chapter 20, Two-Way STOP-Controlled Intersections, contains detailed procedures and discussions for two-way STOP-controlled intersection analysis. Concepts from this chapter, such as critical headways and follow-up times, are applicable to STOP-controlled locations within alternative intersections and interchanges.
- Chapter 34, Interchange Ramp Terminals: Supplemental, contains example problems and origin–destination volume worksheets. Chapter 34 also presents a sketch-planning method for interchange type selection.

2. CONCEPTS

TYPES OF INTERCHANGES AND ALTERNATIVE INTERSECTIONS

Exhibit 23-1 illustrates the various types of interchanges and alternative intersections addressed in this chapter. Note that this exhibit only provides one example of each intersection and interchange type. Many of these intersection and interchange types have several variations, which will be detailed later.

Exhibit 23-1
Types of Intersections and Alternative Interchanges



Note: Gray lines represent the freeway, which is not analyzed as part of the methodology in this chapter.

These interchange and alternative intersection types are as follows:

- *Diamond interchanges:* Ramps from both freeway directions intersect the cross street at separate, but often closely spaced, intersections.
- *Partial cloverleaf (parclo) interchanges:* Parclos contain one or two loop ramps that may intersect the crossroad in a manner similar to a diamond ramp.
- *Diverging diamond interchanges (DDIs):* Similar in configuration to a diamond-type interchange, but with a crossover at each intersection rearranging traffic on the cross street, to reduce conflicts for left-turn movements.
- *Single-point urban interchanges (SPUIs):* All left turns to and from the freeway meet at a single intersection on the cross street.
- *Median U-turn (MUT) intersections:* At-grade intersections at which major- and minor-street left turns are rerouted. Minor-street through movements are not rerouted.
- *Restricted crossing U-turn (RCUT) intersections:* At-grade intersections at which minor-street left-turn and through movements are rerouted. Major-street left turns are not rerouted.
- *Displaced left-turn (DLT) intersections:* At-grade intersections where left-turning vehicles cross opposing through traffic before reaching the main intersection, thus reducing conflicts at the main intersection.

UNIQUE ATTRIBUTES OF INTERCHANGES AND ALTERNATIVE INTERSECTIONS

Interchanges and alternative intersections share a number of characteristics, and they have a number of important differences. Understanding the similarities and differences can be helpful in choosing a configuration or in evaluating operational efficiency. This section describes the common attributes and key differences at such facilities. The common attributes, including origin–destination demands, intersection spacing, signal coordination, demand starvation, and lane utilization on the internal and external approaches, are discussed first.

Influence of Configuration on Turning Movements

The interchange configuration or intersection configuration has a major influence on turning movements. Movements that involve a right-side merge in one configuration may become left turns in another. Movements approaching on the surface facility may be affected by configuration, depending on whether ramp movements involve left or right turns. Thus, the lane utilization on external approaches to the interchange varies as a function of configuration and the relative proportion of turning movements at the downstream intersection.

Influence of Interchange Configurations

In selecting an appropriate type of interchange, impacts on the turning movements should be considered. Left-turning movements are usually the most difficult in terms of efficiency of operation. Left turns from the freeway to the cross street at a signalized diamond interchange, for example, conflict with both directions of the cross street at one ramp terminal and then must traverse the signal at the other ramp terminal. Selection of a type of interchange that does not route high-demand movements through multiple conflict points can enhance the overall flow of vehicles significantly. However, this cannot always be accomplished. Right-of-way limitations or agency policies may preclude the use of loop ramps, and economic and environmental constraints may make multilevel structures impractical. Many of the operational efficiency benefits (as well as safety benefits) derived from alternative interchange designs (such as the DDI) and alternative intersection designs can be attributed to the relocation of high-demand left-turning movements to avoid conflicts.

Because of the influence of interchange type on turning movements and the need to compare various interchange types, LOS for interchange ramp terminals and alternative intersections uses origin–destination (O-D) demands as inputs, since they are identical regardless of the interchange or intersection type. The methodologies in this chapter use both O-D demands and turning movement demands; one set of demands can be derived from the other.

Influence of Alternative Intersection Configurations

The defining characteristic of the alternative intersections presented in this chapter is a rerouting of one or more movements from the center of the intersection, which reduces conflicting flows at the main intersection.

The defining characteristic of the alternative intersections presented in this chapter is a rerouting of one or more movements from the center of the intersection, which reduces conflicting flows at the main intersection. One of the important implications is that the turning movement patterns change from a traditional intersection. For example, at a four-legged RCUT, the minor-street left-turn movement must first turn right at the main junction. Then it must drive to the U-turn crossover, execute a U-turn, drive back to the main junction, and finally make a through movement. At that RCUT, the minor-street left-turn traffic volume will appear in a transformed turning movement diagram as a right turn at the main junction on the near side (closest to the origin), as a U-turn at the crossover, and as a through movement at the main junction on the far side. A similar accounting of all demands must be made for all movements at RCUTs, MUTs, and DLTs. Part C of this chapter shows in detail how to make those transformations for all designs covered.

Rerouting movements at alternative intersections creates junctions in addition to the main junction. At a four-legged MUT, for example, there are three junctions: the main intersection and the ends of the two U-turn crossovers. Vehicles that are rerouted typically must negotiate more than one junction and may experience control delay at each junction. The operational analysis methodology for each type of alternative intersection accounts for each source of control delay for each rerouted movement in the calculation of performance measures and LOS estimates.

At DLT intersections, displaced left-turning vehicles typically experience continuous flow on arriving at the main intersection because they do not yield to opposing through vehicles. Instead, they move together with opposing through vehicles, and signal timing offsets allow them to arrive during a window of available green time. For compatibility with Chapters 18 (Urban Street Segments) and 19 (Signalized Intersections), which do not allow protected left-turning vehicles to move simultaneously with opposing through vehicles, the DLT analysis procedure in Part C assumes that displaced left-turning demand volumes at the main intersection are equal to zero.

Operational Effects of Intersection Spacing

Intersection and interchange configurations with closely spaced signalized intersections present unique challenges because the intersections do not operate in isolation. The distance between intersections can limit the available storage for internally queued vehicles. The presence of a downstream queue may reduce or completely block the discharge from the upstream intersection. Intersection spacing also affects the proportion of vehicles arriving on green and the possibility of unused green time at the downstream intersection.

Queue Interaction

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.

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.

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.

This chapter addresses the interactions of interchange operations with those of adjacent closely spaced signalized intersections. 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, or at other intersection or interchange forms with unsignalized movements. When the queue from a signalized intersection reaches upstream to a roundabout or unsignalized intersection, the upstream operations might be significantly affected. For a roundabout, this queue spillback may cause gridlock because all movements through the roundabout must use the circulating roadway. The HCM computational procedures do not address spillback between interchanges and nearby facilities.

Signal Progression and Lost Time

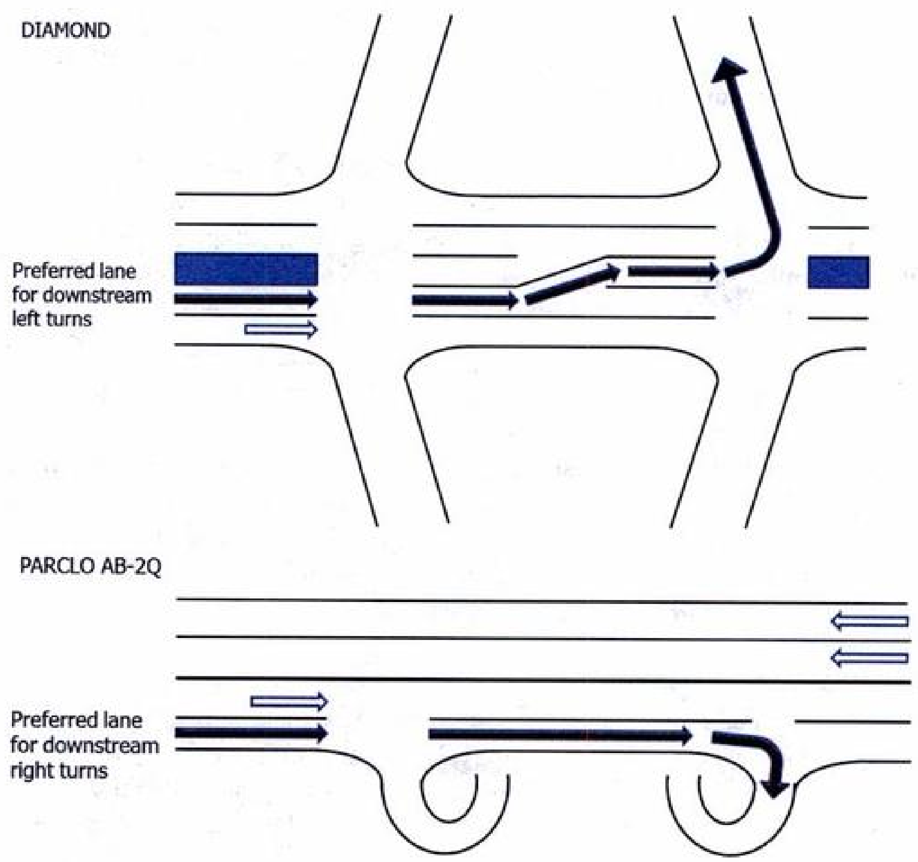
The operational efficiency of distributed intersections is dependent on the spacing of junctions in several other ways. First, junction spacing may allow excellent signal progression through multiple signals, or it may mean that no progression is feasible. Second, facility performance will be affected if vehicles are required to travel longer distances out of their desired paths. Finally, junction spacing may affect signal lost time, such as at a DDI or MUT.

Lane Utilization Effects

Lane utilization is the extent to which lanes are used equally (or unequally) by drivers. The presence of multiple intersections operating as a single unit can strongly influence drivers' choice of lanes when they approach an upstream intersection. At interchanges, this can mean that through-lane utilization at the upstream intersection reflects desired turn movements at the downstream intersection. Likewise, at MUT and RCUT intersections, this can mean that dual right-turn lane utilizations reflect downstream movements, with drivers headed for the U-turn crossover using the leftmost of the side street right-turn lanes.

For two-intersection signalized interchanges, the lane utilization for external through movements approaching the interchange on the surface facility is significantly affected by the direction and demand of turning movements at the downstream intersection. As shown in Exhibit 23-2, significant left-turning demand onto the freeway can lead to highly imbalanced lane utilization on the external approach. Drivers wanting to make a downstream left turn will typically pre-position their vehicles in the leftmost lane(s), in anticipation of the downstream movement. Conversely, heavy-volume downstream right turns will gravitate toward the right side at the upstream intersection. This can create lane-use imbalances exceeding those at single intersections.

Exhibit 23-2
Impact of Interchange Type on Lane Utilization



This chapter's methodology identifies the highest lane utilization 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. 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.

Traffic Control Considerations

When multiple controls (signals, YIELD signs, STOP signs) are present within a single intersection or set of intersections, a hybrid analysis may be required. Such an analysis should utilize principles of the unsignalized and signalized HCM chapters (Chapters 16 through 24) and should work toward obtaining a common performance measure. Although unsignalized movements can occur at signalized intersections, hybrid control combinations are more common at certain distributed configurations, such as DDIs, RCUTs, and MUTs. At DDIs, critical headways and follow-up times affect the analysis of free-flow and YIELD-controlled turns. Some distributed intersections have traffic movements that could be considered unconventional. For example, U-turn movements must be made at RCUT and MUT intersections to reach certain destinations from certain origins. These movements produce the need for specific analytical techniques, which are described in Parts B and C of this chapter.

At distributed intersections with signals, the signal operations differ from conventional intersections in some specific ways. Signal operations at distributed intersections are different in that they can provide, and rely heavily on, a maximum of one stop along the major arterial. This minimizes delay and effectively manages the length and location of vehicle queues. If an alternative intersection junction is signalized, the signal will almost always have two phases. The common exceptions to this are a U-turn crossover signal serving a busy two-way minor street or driveway and the main signal at a partial DLT, where the street without left-turn crossovers may need an exclusive left-turn phase. The multiple signals that are below capacity should have the same cycle length (or a variation such as half-cycle) to allow progression. At a DLT, the left-turn crossover intersection almost always has an offset relative to the signal at the main junction that allows most through drivers on that street to arrive on green. At an MUT, the main junction almost always has an offset relative to the U-turn crossover that allows most through drivers on that street to arrive on green. At an RCUT, signals on each side of the major street can be timed independently, with different cycle lengths if desired. But, along each side of the major street, signal offsets typically allow maximum bandwidths for through traffic.

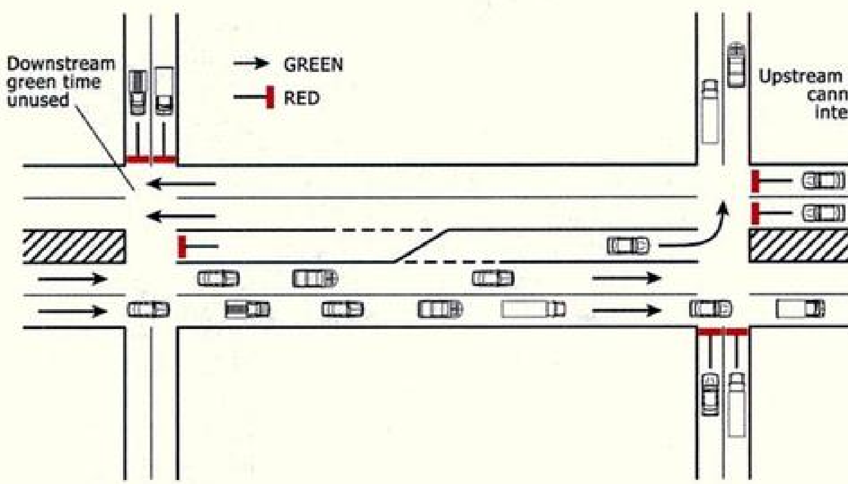
Signal operations at distributed intersections are different in that they can provide, and rely heavily on, a maximum of one stop along the major arterial. This minimizes delay and effectively manages the length and location of vehicle queues.

Compared with conventional signalized intersection analysis, the important modeling differences in this chapter relate to lane utilization, saturation flow rate, and signal progression. Lane utilization at RCUT and MUT intersections may differ from that at conventional intersections, because some drivers pre-position themselves at one junction to get ready for the second junction. Saturation flow rates for U-turns at RCUTs and MUTs differ from those for left turns at conventional intersections. Signal progression is an important feature at RCUTs, MUTs, and DLTs, as agencies attempt to progress large portions of through movements through multiple signals. The methodology presented in Part C takes all of these important operational characteristics into account in the estimation of performance measures and LOS.

Demand Starvation at Conventional Interchanges

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 23-3 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. 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 23-3
Demand Starvation at the Internal Link of a Diamond Interchange



COMPARING INTERCHANGE AND INTERSECTION EVALUATIONS

To follow up on the prior discussion of unique attributes of distributed intersections, Exhibit 23-4 summarizes their shared elements and differences. Exhibit 23-4 shows that multiple interchange types require specific adjustments for lane utilization, saturation flow rate, critical headway, and lost time. Conversely, some configuration type evaluations have their own unique adjustments. These include the assumption of zero displaced left-turning demands at the main intersection (DLT), U-turn lost time and saturation flow rate adjustments (RCUT, MUT), and left turns on red (DDI). Parclo and diamond interchanges are seen to have the same basic framework for evaluation. The SPUI modeling framework is a subset of the parclo and diamond framework. These modeling frameworks are detailed in Part B (Interchange Ramp Terminal Evaluation) and Part C (Alternative Intersection Evaluation).

Exhibit 23-4
Summary Comparison of Intersection and Interchange Procedures

Adjustments Beyond Standard Intersection Analysis	Diamond	Parclo	SPUI	DDI	DLT	RCUT, MUT
Volume Adjustment						
Assume displaced left-turn demand volume of zero at the main intersection					Y	
Lane Utilization Adjustments						
Upstream (external) through movements	Y	Y	Y	Y		D
Minor-street turning movements						D
Unsignalized Control-Based Adjustments						
Critical headway, follow-up time: right turn on red	Y	Y	Y	Y		Y
Critical headway, follow-up time: left turn on red				Y		
Critical headway, follow-up time: U-turn on red						Y
Yielding right turn			(Addressed in Chapter 18)			
Yielding U-turn						Y
Signalized Control-Based Adjustments						
Saturation flow rate: traffic pressure	Y	Y	Y	Y		
Saturation flow rate: turn radius	Y	Y	Y	Y	Y	Y
Saturation flow rate: U-turn						Y
Saturation flow rate: DDI				Y		
Yielding left turn				Y		
Downstream link queue: internal	Y	Y		Y		
Demand starvation: internal	Y	Y		Y		
Downstream link queue: external	Y	Y		Y		
Demand starvation: external	Y	Y		Y		
Additional all-red/lost time	Y	Y	Y	Y		
Specially designed offset(s)					Y	D
Other Adjustments						
Weave/merge adjustments						Y

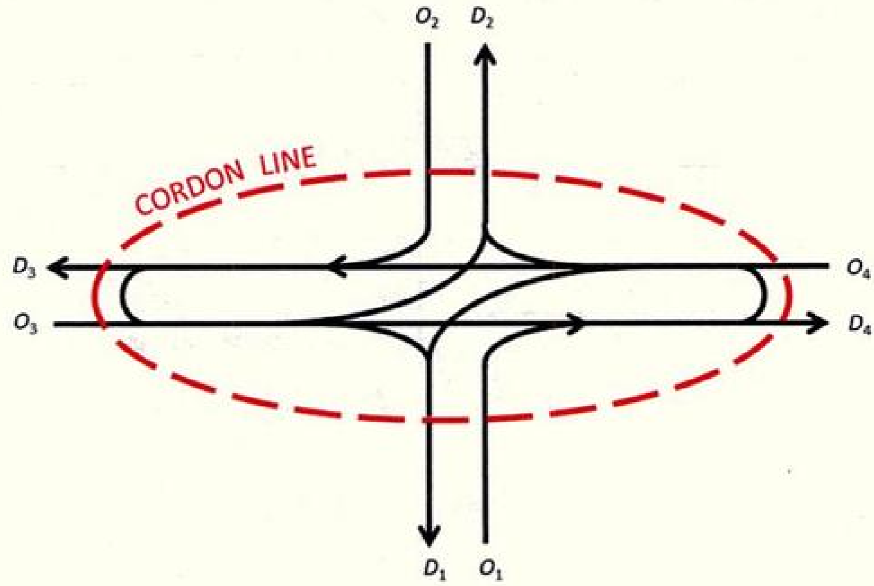
Notes: Parclo = partial cloverleaf, SPUI = single-point urban interchange, DDI = diverging diamond interchange, DLT = displaced left turn, RCUT = restricted crossing U-turn, and MUT = median U-turn.
Y = adjustment generally applicable and D = applicability depends on site configuration.

SPATIAL AND TEMPORAL LIMITS

Distributed intersections are closely spaced, interdependent intersections best analyzed as a single unit. When the spatial limits of the analysis are defined, a cordon line is drawn around the area to be studied. For example, at the RCUT intersection shown in Exhibit 23-5, the U-turn intersections essentially serve as spatial analysis boundaries along the signalized arterial. However, if the cordon line were to include nearby intersections beyond those U-turn locations, the Chapter 23 procedures might not be able to analyze the full area, and alternative tools might be needed.

Despite having multiple junctions, distributed intersections act as single nodes with only three or four legs, origins, and destinations. The origin (O_1, O_2, O_3, O_4) and destination (D_1, D_2, D_3, D_4) points shown in Exhibit 23-5 are generally applicable to most intersections and interchanges described in this chapter. The O-D methodologies outlined in Parts B and C are based on this concept.

Exhibit 23-5
Example of Spatial Limits for an RCUT Intersection



With regard to the temporal limits of the analysis, the HCM typically recommends modeling the peak 15-min period. Multiple-period analyses are commonly used to study oversaturated conditions and to study residual queues persisting from one time period to the next. However, the Chapter 23 procedures do not address queue spillback or queue spillover. Thus, alternative tools might be required when significant oversaturation exists.

LOS FRAMEWORK

Chapter 23 requires an LOS framework capable of capturing specific signalized and arterial operations in a way that facilitates unbiased comparisons among types of distributed intersections.

In developing an LOS framework for distributed intersections, consideration of existing frameworks for similar facilities is informative. For isolated signalized intersections (Chapter 19), average control delay per vehicle is an intuitive measure for LOS determination. For urban street segments (Chapter 18), the average difference between free-flow and actual speed is a fundamental quality-of-service indicator. Chapter 23 requires an LOS framework capable of capturing specific signalized and arterial operations in a way that facilitates unbiased comparisons among types of distributed intersections.

Control delay would not be suitable as the sole measure for determining LOS (as in Chapter 19), since it would not account for the diverted-path delay present at some facilities. Travel speed would not be suitable as the service measure (as in Chapter 18), because it does not describe the efficiency of sequential major- and minor-street movements. Instead, the distributed intersections are all responsible for a certain amount of *experienced travel time*. More specifically, each O-D path can experience (*a*) *control delay* at signalized or unsignalized locations and (*b*) *extra distance travel time*. Some O-D paths may have multiple instances of one or more of these elements. These elements can be used together to determine the *experienced travel time*, and from this the performance measures of Chapter 23 can be derived. Equation 23-1 can be used to compute experienced travel time (*ETT*):

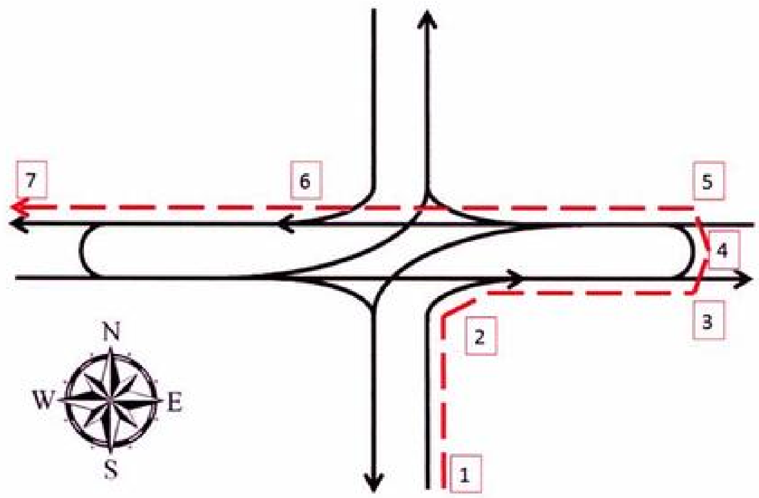
$$ETT = \sum d_i + \sum EDTT$$

Equation 23-1

where d_i is the control delay at each junction i encountered on the path through the facility and $EDTT$ is the extra distance travel time.

Exhibit 23-6 illustrates the concept of providing unbiased comparisons among distributed intersection configurations by using an RCUT intersection example. The dashed line denotes the path of a typical left turner arriving from the minor street and entering the major street. Summarizing control delays in accordance with Chapter 19 at all three intersections (i.e., westmost, middle, eastmost) would not capture diverted-path travel times between Points 2 and 6. Furthermore, average travel speeds (in accordance with Chapter 18) in the east–west arterial directions would not consider control delays at Points 2 and 4. An unbiased comparison between configurations would require consideration of experienced travel times between all O-D points encircling the system, with the system and O-D points spatially defined as in Exhibit 23-5.

Exhibit 23-6
Example of Experienced Travel Time at an RCUT Intersection



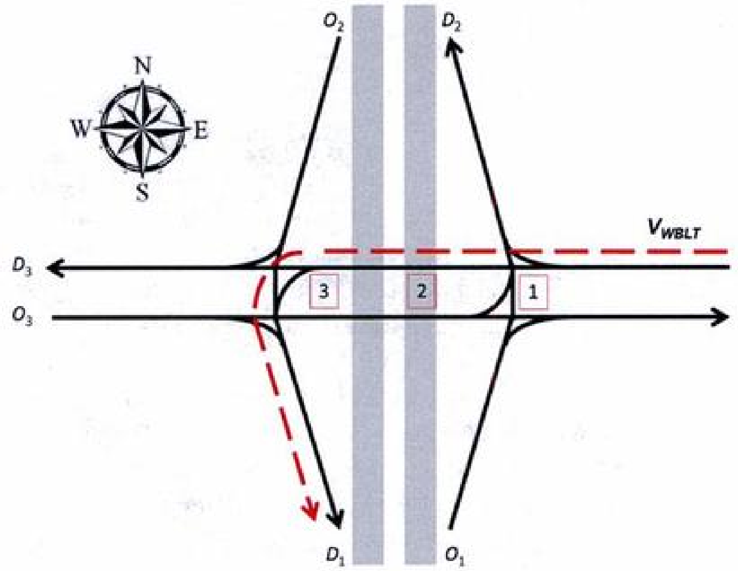
Signalized Interchanges

The LOS designation is based on the operational performance of O-D demands (shown in Exhibit 23-7) through the interchange. The LOS for each O-D is based on the average experienced travel time ETT of that demand as it travels through the interchange. For example, for the diamond interchange shown in Exhibit 23-7, ETT for O-D movement O_4-D_1 is equal to the sum of westbound through control delay at Point 1 (d_{WBT1}), control delay at Point 3 (d_{WBL3}), and extra distance travel time that lies roughly between Points 2 and 3 ($EDTT_{23}$). Thus, the ETT could be expressed as follows:

$$ETT_{41} = d_{WBT1} + d_{WBL3} + EDTT_{23}$$

Equation 23-2

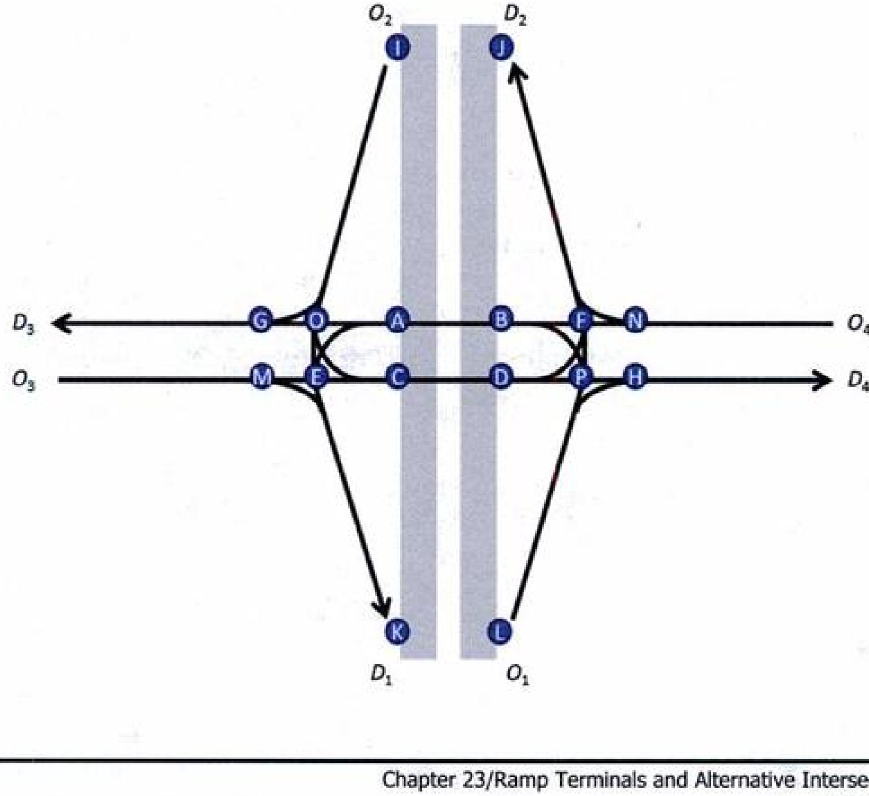
Exhibit 23-7
Illustration of the LOS Concept at a Diamond Interchange



To compute the exact EDTT for this O-D movement, free-flow travel time beyond Point 2 would be compared with the (hypothetical) free-flow travel time that would occur if it were possible to turn left immediately on reaching the southbound freeway. This EDTT would be similar to, but not necessarily identical to, the free-flow travel time between Points 2 and 3.

Exhibit 23-8 illustrates the EDTT calculations given by Equation 23-3 through Equation 23-10 for various O-D movements. EDTT subtracts hypothetical-path free-flow travel times, which would occur under 90-degree (i.e., right angle) turns, from actual-path free-flow travel times. This calculation produces positive EDTTs for all left turns and negative EDTTs for all right turns.

Exhibit 23-8
Illustration of the EDTT Concept at a Diamond Interchange



$$EDTT_{13} = TT_{LPB} - TT_{LB}$$ **Equation 23-3**

$$EDTT_{14} = TT_{LH} - TT_{LDH}$$ **Equation 23-4**

$$EDTT_{24} = TT_{IOC} - TT_{IC}$$ **Equation 23-5**

$$EDTT_{23} = TT_{IG} - TT_{IAG}$$ **Equation 23-6**

$$EDTT_{32} = TT_{DFJ} - TT_{DJ}$$ **Equation 23-7**

$$EDTT_{31} = TT_{MK} - TT_{MCK}$$ **Equation 23-8**

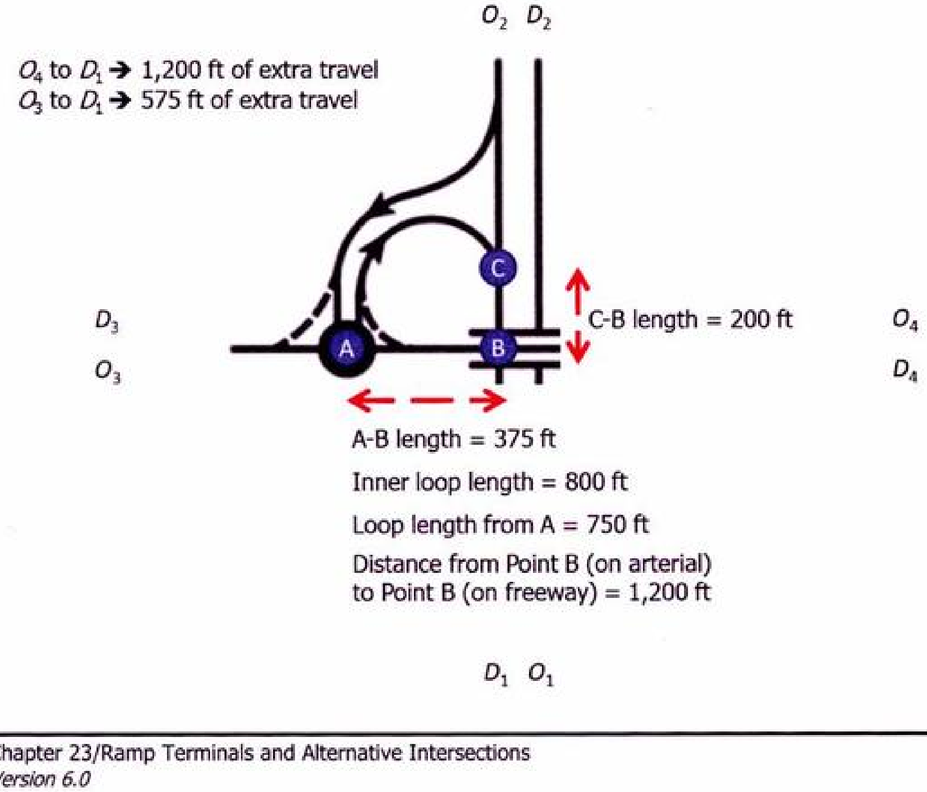
$$EDTT_{41} = TT_{AEK} - TT_{AK}$$ **Equation 23-9**

$$EDTT_{42} = TT_{NJ} - TT_{NBJ}$$ **Equation 23-10**

where $EDTT_{13}$ is the extra distance travel time between origin 1 and destination 3, TT_{LPB} is the travel time along path L-P-B in Exhibit 23-8, TT_{LB} is the travel time along (hypothetical) path L-B, other numbered subscripts indicate other origin–destination pairs, and other lettered subscripts indicate other paths through the interchange.

Exhibit 23-9 illustrates the concept of EDTT calculation at a Parclo A-2Q interchange. Vehicles traveling from origin O_4 to destination D_1, instead of being able to turn directly left at Point B, experience a full 1,200 ft of out-of-direction travel beginning at Point B along the arterial and ending at Point B along the freeway. In contrast, vehicles traveling from origin O_3 to destination D_1 only experience 750 + 200 – 375 = 575 ft of out-of-direction travel (i.e., along-the-loop-ramp distance, plus end-of-loop-to-overpass distance, minus intersection-to-overpass distance).

Exhibit 23-9
Illustration of the EDTT Concept at a Parclo A-2Q Interchange



The first column of Exhibit 23-10 summarizes the delay-based LOS criteria for each O-D within signalized interchanges. The second and third columns of Exhibit 23-10 show that LOS F is defined to occur when either the volume-to-capacity (v/c) ratio or the average queue-to-storage ratio (R_Q) for any of the lane groups that contain this O-D exceeds 1. Storage is defined as the distance available for queued vehicles on a particular movement, and it is measured on a per lane basis. For example, if the left-turning lane group shown in Exhibit 23-7 has $v/c > 1$, the LOS for the entire O-D movement O_4-D_1 will be F. If a particular lane group has $v/c > 1$, 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, all O-Ds traveling through this lane group will operate at LOS F, regardless of their delay.

The values presented in Exhibit 23-10 reflect a control delay component greater by a factor of 1.5 than those for signalized intersections. This reflects the need for O-D movements to travel through multiple intersections.

Exhibit 23-10
LOS Criteria for Each O-D Within Signalized Interchanges

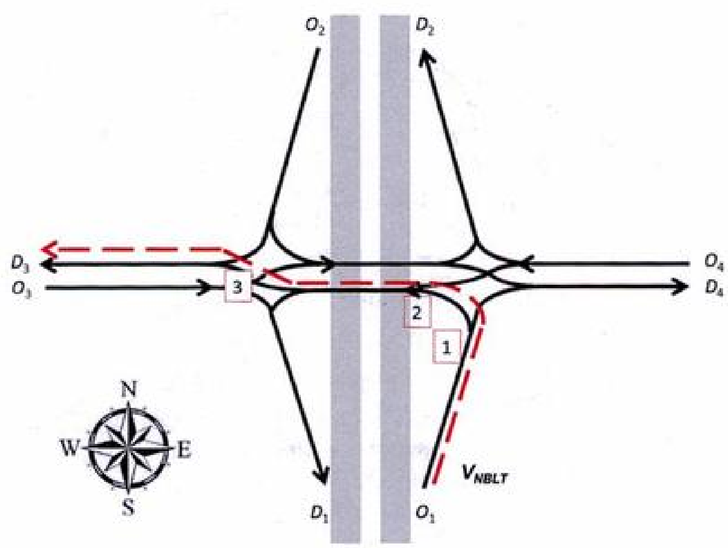
	Condition		
ETT (s/veh)	$v/c \leq 1$ and $R_Q \leq 1$ for Every Lane Group	$v/c > 1$ for Any Lane Group	$R_Q > 1$ for Any Lane Group
≤15	A	F	F
>15–30	B	F	F
>30–55	C	F	F
>55–85	D	F	F
>85–120	E	F	F
>120	F	F	F

As an illustration, consider the DDI shown in Exhibit 23-11. The ETT for O-D movement O_1-D_3 is equal to the sum of northbound left-turn control delay at Point 2 (d_{NBL2}), westbound through control delay at Point 3 (d_{WBT3}), and extra distance travel time between Points 1 and 2 ($EDTT_{12}$). Thus, the ETT for a northbound left-turn movement (originating at the northbound freeway off-ramp) could be expressed as follows:

Equation 23-11

$$ETT_{13} = d_{NBL2} + d_{WBT3} + EDTT_{12}$$

Exhibit 23-11
Illustration of the ETT Concept at a DDI



Alternative Intersections

As with signalized interchanges, the LOS for alternative intersections is based on the operational performance of O-D demands through the intersections (previously shown in Exhibit 23-6). LOS for each O-D movement is again based on the average ETT for that demand as it travels through the intersections. In displaced left-turn cases where the extra distance travel times are negligible, the ETT is equivalent to the sum of control delays, as shown in Equation 23-12. For example, for the DLT intersection shown in Exhibit 23-12, the ETT for O-D movement O_4-D_1 is equal to the westbound left-turn average control delay at Point 1, plus the southbound through average control delay at Point 2, applied to the flow rate traveling from O_4 to D_1. Movement O_4-D_1 can also be described as the westbound left-turn movement for the DLT intersection as a whole. Thus, ETT for the movement from origin O_4 to destination D_1 is as follows:

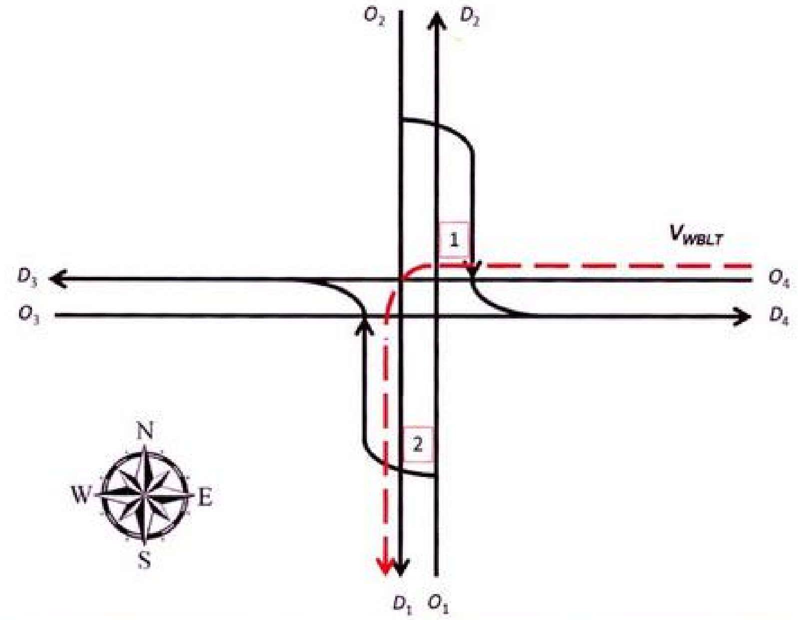


Exhibit 23-12
Illustration of the ETT Concept at a Displaced Left-Turn Intersection

Note: Southbound through delay at Point 2 could be zero if designed for a protected-only phase at Point 1.

$$ETT_{41} = d_{WBL1} + d_{SBT2}$$

Equation 23-12

Exhibit 23-13 summarizes the LOS criteria for each O-D movement within alternative intersections. The values presented in Exhibit 23-13 reflect control delay thresholds identical to those for conventional signalized intersections and 33% lower than those for interchanges (i.e., in Exhibit 23-10).

Exhibit 23-13
LOS Criteria for Each O-D Within Alternative Intersections

	Condition		
ETT (s/veh)	$v/c \le 1$ and $R_Q \le 1$ for Every Lane Group	$v/c > 1$ for Any Lane Group	$R_Q > 1$ for Any Lane Group
≤10	A	F	F
>10–20	B	F	F
>20–35	C	F	F
>35–55	D	F	F
>55–80	E	F	F
>80	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 movement is based on the total average control delay experienced by that demand as it travels through the interchange. Exhibit 23-14 summarizes the LOS criteria for each O-D of an interchange with one or two roundabouts. The values presented in Exhibit 23-14 are greater than those for non-interchange 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 values for signalized interchanges, since drivers would likely expect lower delays at roundabouts.

Exhibit 23-14
LOS Criteria for Each O-D of an Interchange with Roundabouts

	Condition		
ETT (s/veh)	$v/c \leq 1$ and $R_Q \leq 1$ for All Approaches	$v/c > 1$ for Any Approach	$R_Q > 1$ 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 movement, along with other applicable performance measures. These performance measures can be determined with procedures in this and other HCM chapters, alternative tools, or both, aggregated as appropriate into O-D performance measures by using the techniques in this chapter.

Part B: Interchange Ramp Terminal Evaluation

1. INTRODUCTION

OVERVIEW

Interchange ramp terminals are critical components of the highway network. They provide the connection between various highway facilities (freeway–arterial, arterial–arterial, etc.), and thus their efficient operation is essential. Interchanges must 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.

This section presents the methodology for the analysis of interchanges involving freeways and surface streets (i.e., service interchanges). It was developed primarily on the basis of research conducted through the National Cooperative Highway Research Program (*1–3*), Texas Department of Transportation research (*4*), and FHWA research on DDIs (*5*).

PART ORGANIZATION

This part of Chapter 23 presents methodologies for the evaluation of interchanges, including diamond interchanges, DDIs, SPUIs, interchanges with roundabouts, and parclo interchanges. Section 2 presents additional concepts specific to interchanges not covered in Part A. Section 3 presents the core methodology for evaluating the operational performance of interchanges for diamond interchanges, DDIs, SPUIs, and parclos. Section 4 provides extensions to the methodology, including evaluation of interchanges with roundabouts and other unsignalized intersections, and a discussion of pedestrian and bicycle analysis at interchanges. Section 5 provides applications of the methodology, including example results, a discussion of types of analysis, and considerations for the use of alternative tools at interchanges.

2. CONCEPTS

TYPES OF INTERCHANGES

A number of types of interchanges are recognized in the literature. *A Policy on Geometric Design of Highways and Streets* (*9*) provides extensive information on interchange designs and their characteristics. Part A discussed intersections and interchanges in broad terms; this section more specifically illustrates and discusses the interchange designs considered in this chapter: diamond interchanges, DDIs, SPUIs, parclos, and interchanges with roundabouts.

Diamond Interchanges

Most forms of diamond interchanges result in two or more closely spaced surface intersections, as illustrated in Exhibit 23-15. 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. 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.

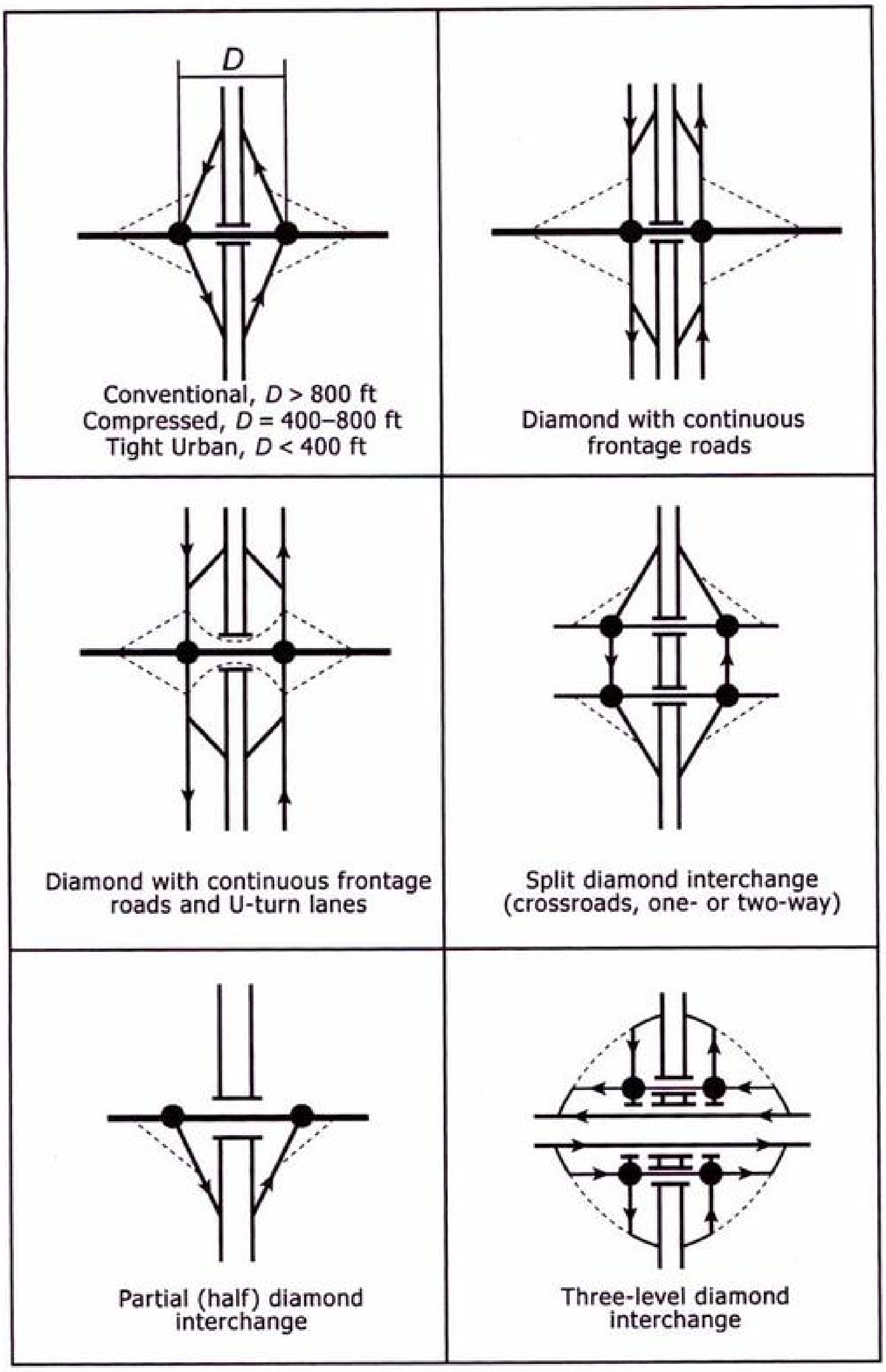
The interchange methodology is only applicable when both ramp terminals are signalized or both are roundabouts.

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 and tight 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.

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 and U-turns removed 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 23-15. 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.

Exhibit 23-15
Types of Diamond Interchanges

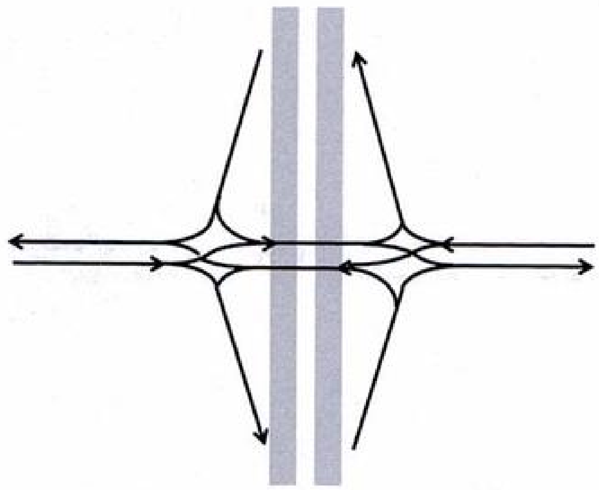


Note: ------ Possible alternative configuration of signal bypasses operating as unsignalized movements; these are movements that are not using the ramp terminals.

Diverging Diamond Interchange

The DDI, sometimes also referred to as a double crossover diamond interchange, is a variation of the traditional diamond interchange at which the cross-street movements cross directions twice, once at each ramp terminal. Except for potential pedestrian conflicts, the DDI allows left turns onto the freeway to be a free-flowing movement at the internal crossover, which results in large efficiency gains for interchanges with heavy left-turn demand. Left turns from the freeway are similarly able to merge onto the arterial street in their desired direction of travel without crossing over the opposing through movement. A schematic of a DDI in east–west orientation is shown in Exhibit 23-16.

Exhibit 23-16
Illustration of a DDI

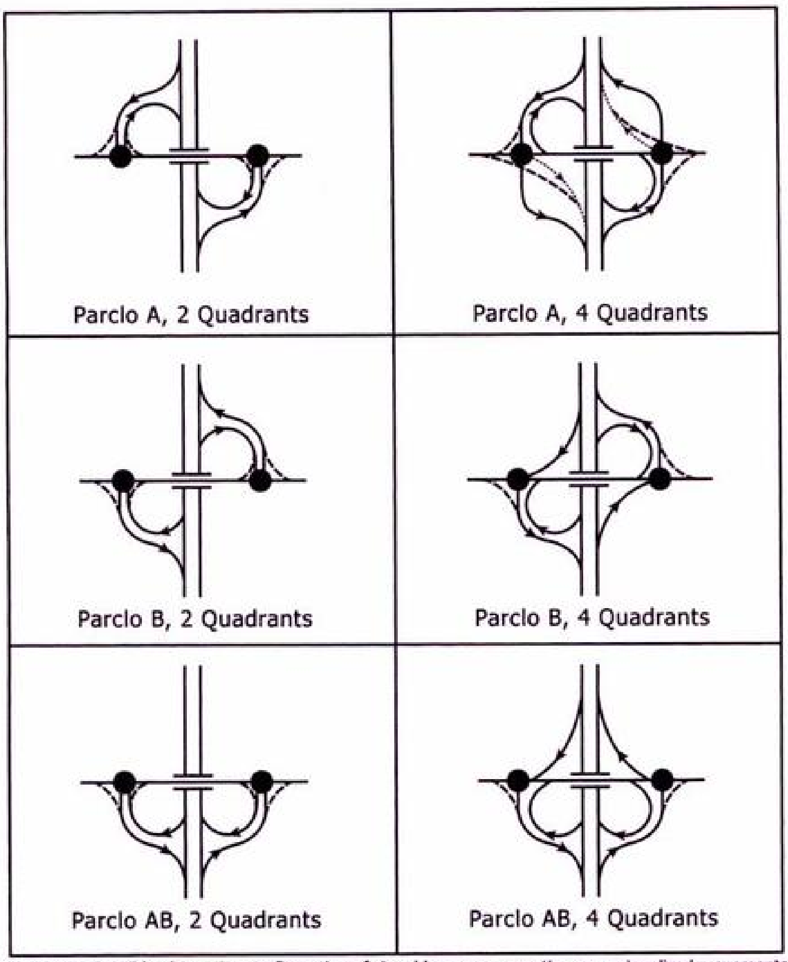


Research on the operational performance of the DDI (*6–8*) showed that despite its potential for enhancing the operational efficiency of a standard diamond interchange (especially for locations with heavy left-turning movements), several unique considerations apply in an operational evaluation of the DDI. Among them are lane utilization at the external crossover, saturation flow rate adjustments for the crossover movement, lost-time considerations for through and turning movements, and capacity of YIELD-controlled turns. The methodology described in this chapter accounts for these operational characteristics of DDIs.

Parclo Interchanges

Parclo interchanges are shown in Exhibit 23-17. A variety of parclo interchanges can be created with one or two loop ramps. In such cases, one or two of the outer ramps intersect the crossroad in a manner similar to 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 23-17
Types of Parclo Interchanges



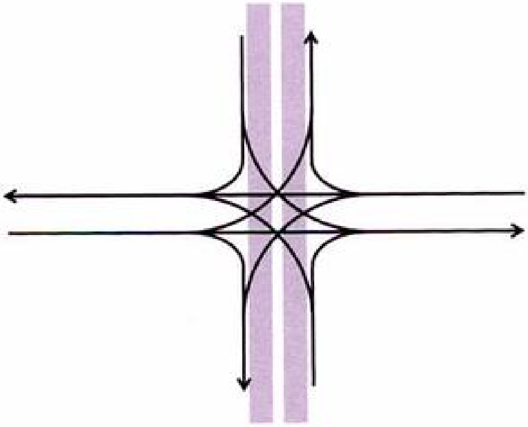
Note: ----- Possible alternative configuration of signal bypasses operating as unsignalized movements; these are movements that are not using the ramp terminals.
.... Possible alternative configuration of right-turn-only ramp passing through the ramp terminal.

In Parclo A forms, loop ramps on the mainline occur in advance of the crossroad. In Parclo B forms, loop ramps on the mainline occur beyond the crossroad. In Parclo AB forms, loop ramps on the mainline are located on the same side of the crossroad, one in advance of the crossroad for its direction of travel and the other beyond.

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 23-18.

Exhibit 23-18
Single-Point Urban Interchange

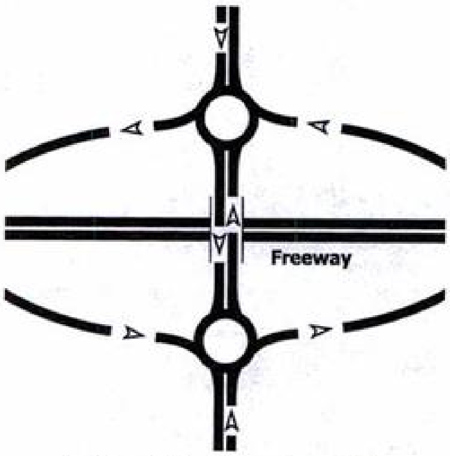


Interchanges with Roundabouts

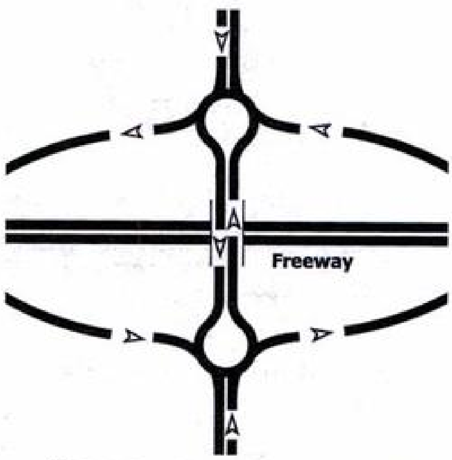
Roundabouts can replace signalized or STOP-controlled intersections as interchange ramp terminals. Three types of roundabout ramp terminal designs are typically used in the United States and are illustrated in Exhibit 23-19. The first design consists of two traditional roundabouts at the two nodes of 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. These two designs are essentially the same, except that the former should be provided when U-turns are allowed or when there is an additional approach to the roundabout. The third 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, which significantly reduces 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.

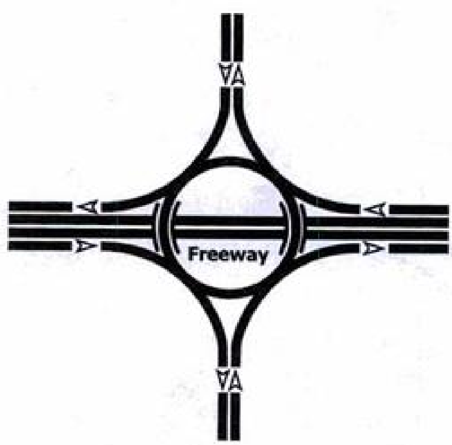
Exhibit 23-19
Diamond Interchanges with Circular Ramp Terminals



(a) Roundabout Ramp Terminals



(b) Raindrop-Shaped Ramp Terminals

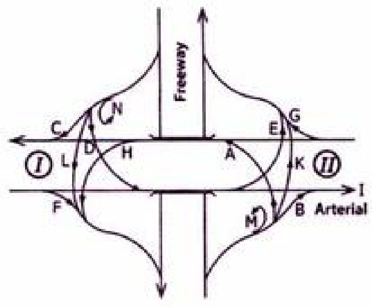


(c) Single-Point Roundabout Interchange

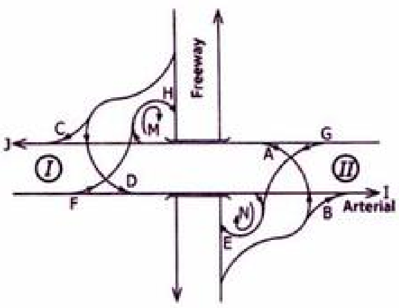
O-D AND TURNING MOVEMENTS FOR CONVENTIONAL INTERCHANGES

Exhibit 23-20 illustrates the O-D movement letters for different types of interchanges considered in this methodology. In Chapter 34, Interchange Ramp Terminals: Supplemental, Exhibit 34-163 through Exhibit 34-178 provide the corresponding calculations to obtain turning movements from O-D movements and vice versa.

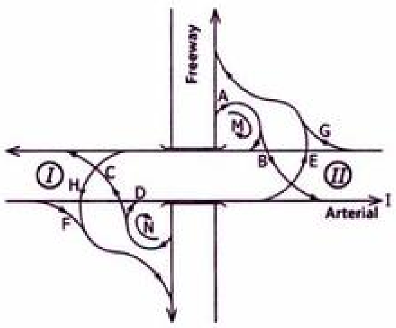
Exhibit 23-20
O-D Flows for Each Interchange Configuration



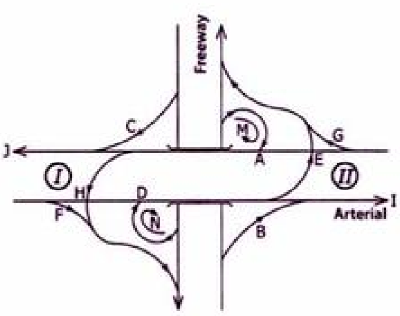
(a) Diamond Interchange



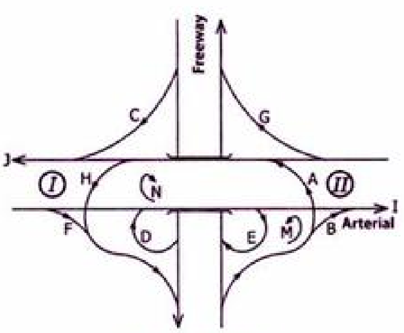
(b) Parclo A-2Q



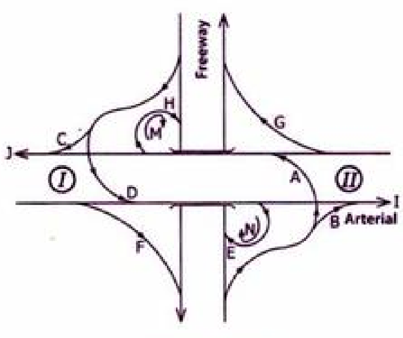
(c) Parclo B-2Q



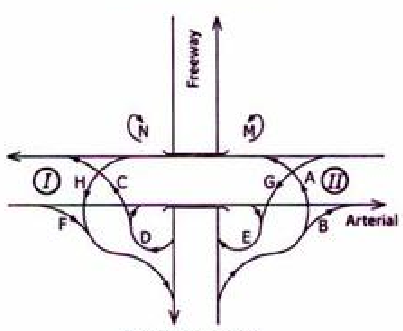
(d) Parclo B-4Q



(e) Parclo AB-4Q



(f) Parclo A-4Q



(g) Parclo AB-2Q

3. CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

Spatial and Temporal Limits

The methodology addresses interchanges with signalized intersections, interchanges with roundabouts, and the impact and operations of adjacent closely spaced intersections. The methodology also addresses DDIs with both signalized and unsignalized turning movements. 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 diamonds, DDIs, SPUIs, and parclos. The methodology addresses at-grade intersections, not including the freeway proper, and focuses on surface streets; it does not analyze freeway-to-freeway interchanges.

Chapter 34, Interchange Ramp Terminals: Supplemental, includes a methodology for assessing the operational performance of various types of interchanges for purposes of interchange type selection. The method 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 non-operational concerns.

Performance Measures

The operational analysis methodology for interchanges provides the performance measure *experienced travel time* (ETT). For each movement, the ETT includes the control delay experienced at each junction encountered, plus the time experienced in traveling any extra distances required by the design. It may be expressed as follows:

Equation 23-13

$$ETT = \sum d_i + \sum EDTT$$

where d_i is the control delay at junction i encountered on the path through the interchange (seconds) and $EDTT$ is the extra distance travel time (seconds).

The methodology computes control delays at each individual junction making up the interchange, so useful related measures such as capacity and v/c ratio are available for each of those junctions. Use of the ETT performance measure allows comparison of interchanges of different forms on the same basis. Intersections (conventional and alternative) may be compared with interchanges having multiple junctions or with rerouted movements driving longer distances.

Standard diamond and parclo interchanges have non-zero EDTT values because their travel paths deviate from the freeway centerline. In these cases, EDTT is calculated at the ramp design speed.

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 of 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.
- The HCM provides deterministic estimates of the measures of effectiveness, which is important for some purposes such as development impact review.

Limitations of the Methodology

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 when the downstream queue spills back into the upstream intersection for long periods of time;
- The impact of spillover into adjacent travel lanes due to inadequate turn-pocket length;
- 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;
- Interchanges with two-way STOP-controlled intersections or interchanges consisting of a signalized intersection and a roundabout;
- Lane utilizations for interchanges with additional approaches that are not part of the prescribed interchange configuration (however, guidance is provided for addressing those cases);
- Lack of provision of link travel times and speeds (the methodology does provide delay estimates); 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 the user is interested in the analysis of conditions that fall within 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 5 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 interchanges.

Alternative Tool Considerations

General guidance on 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. Section 5 of this part contains more 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 Section 5 will be limited to microsimulation tools.

This chapter provides a methodology for estimating the capacity, control delay, queue storage, and LOS for a given set of traffic, control, and design conditions at an interchange. As with most 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 performance measures such as number of stops, fuel consumption, and pollution. They are also useful for the evaluation of other modes, including pedestrians, cyclists, and transit, and their interaction with vehicles at interchanges. The animated graphics offered by many simulation tools are especially useful for observing network operations and identifying problems at specific elements.

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, Interpreting HCM and Alternative Tool Results, discusses simulation-based performance measures in more detail.

REQUIRED DATA AND SOURCES

The analysis begins with the assembly of all pertinent input data, such as geometric characteristics, traffic demands, and signalization information. Exhibit 23-21 provides a summary of all input data required in conducting an operational analysis for interchange ramp terminals.

Exhibit 23-21
Summary of Required Input Data for Final Design and Operational Analysis of Signalized Interchanges

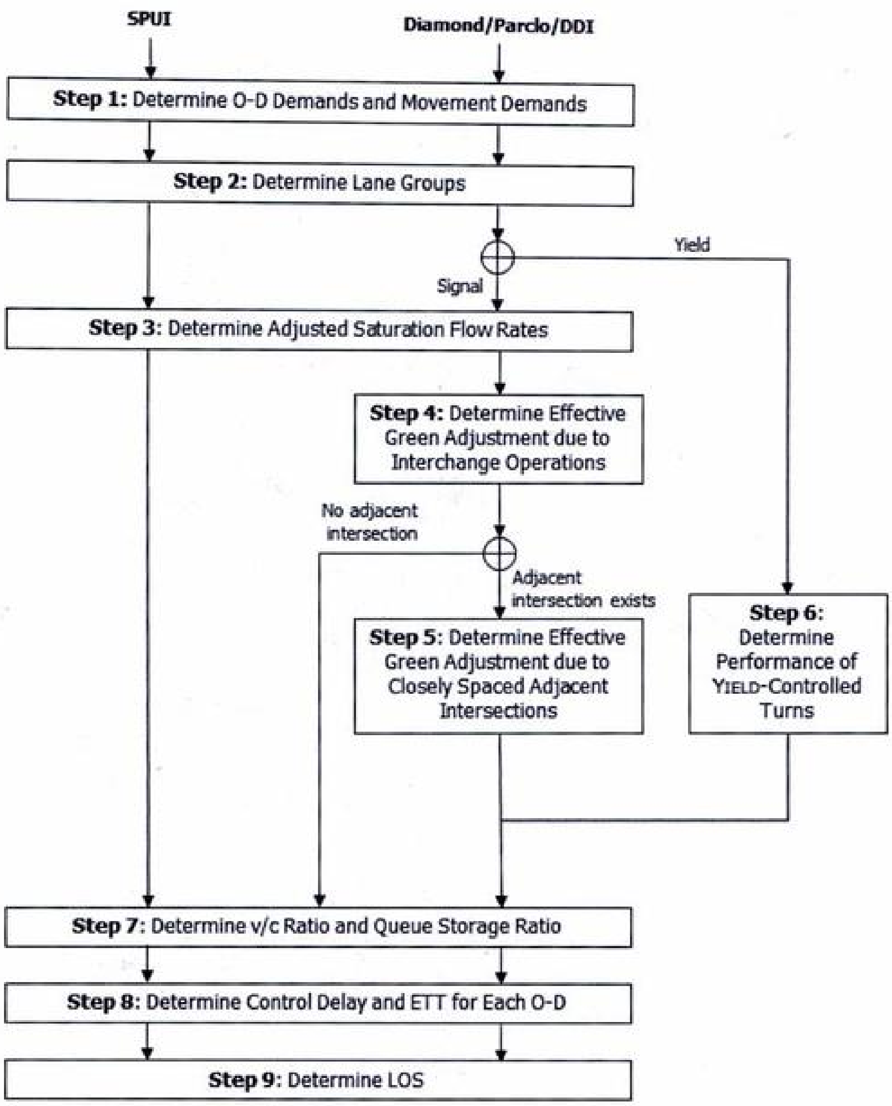
Parameter	Potential Data Source	Suggested Default Value
	Geometric Data	
Area type	Field data, aerial photo	Must be provided
Number of lanes (*N*)	Field data, aerial photo	Must be provided
Average lane width (*W*, ft)	Field data, aerial photo	12 ft
Grade (*G*, %)	Field data	Flat approach: 0% Moderate grade: 3% Steep grade: 6%
Existence of exclusive left- or right-turn lanes	Field data, aerial photo	Must be provided
Length of storage for each lane group (L_a, ft)	Field data, aerial photo	Must be provided
Distance between the two intersections in the interchange (*D*, ft)	Field data, aerial photo	Must be provided
Distances corresponding to the internal storage between interchange intersections and nearby adjacent intersections (ft)	Field data, aerial photo	Must be provided
Turning radii for all turning movements (ft)	Field data, aerial photo	Must be provided
Extra travel distance relative to centerline (ft)	Field data, aerial photo	Must be provided
	Demand and Traffic Data	
Demand volume by O-D or turning movement (*V*, veh/h)	Field data, past counts	Must be provided
Right-turn-on-red flow rates	Field data, past counts	0.0 veh/h
Left-turn-on-red flow rates	Field data, past counts	0.0 veh/h
Base saturation flow rate (s_0, pc/hg/ln)	Field data, judgment	Metro pop. ≥ 250,000: 1,900 pc/h/ln Otherwise: 1,750 pc/h/ln
Peak hour factor (*PHF*)	Field data, judgment	Total entering vol. ≥ 1,000 veh/h: 0.92 Total entering vol. < 1,000 veh/h: 0.90
Percent heavy vehicles (*HV*, %)	Field data, past counts	3%
Approach pedestrian flow rates (v_{ped}, p/h)	Field data, past counts	Must be provided
Approach bicycle flow rates (v_b, bicycles/h)	Field data, past counts	Must be provided
Local bus stopping rate (N_B, buses/h)	Field data, judgment	Central business district (CBD) bus stop: 12 buses/h Non-CBD: 2 buses/h
Parking activity (N_m, maneuvers/h)	Field data, judgment	Must be provided
Arrival type (*AT*)	Field data, judgment	3
Upstream filtering adjustment factor	Field data, judgment	1.0
Approach speed (S_A, mi/h)	Field data, judgment	Speed limit + 5 mi/h
Free-flow speed (S_f, mi/h)	Field data, judgment	Speed limit + 5 mi/h
	Signal Data	
Type of signal control	Field data	Must be provided
Phase sequence	Field data	Must be provided
Cycle length (if appropriate) (*C*, s)	Field data	Must be provided
Green times (if appropriate) (*G*, s)	Field data	Must be provided
Yellow-plus-all-red change-and-clearance interval (intergreen) (*Y*, s)	Field data	Must be provided
Offset (if appropriate)	Field data	Must be provided
Maximum, minimum green, passage times, phase recall (for actuated control)	Field data	Must be provided
Presence of pedestrian push button	Field data	Must be provided
Minimum pedestrian green (G_p, s)	Field data	Must be provided
Phase plan	Field data	Must be provided

Agencies that use the methodologies of this chapter are encouraged to develop a set of local default values (where defaults are applicable) 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 Exhibit 23-21 can be used if appropriate.

OVERVIEW

Exhibit 23-22 summarizes the basic nine-step methodology for the design and operational analysis of signalized interchange ramp terminals. The methodology is similar to that of Chapter 19, Signalized Intersections, with further 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 23-22
Interchange Ramp Terminals Methodology: Final Design and Operational Analysis for Interchanges



Note: Step 6 also includes determination of performance of free-flow turn movements.

The analysis of SPUIs is outlined on the left part of the flowchart. The flowchart highlights only the components added to the signalized intersection methodology for analyzing SPUIs. The right part of the flowchart highlights the components added to the signalized intersection methodology for analyzing diamond, diverging diamond, and parclo interchanges. Each of the steps outlined in Exhibit 23-22 is explained and discussed below.

COMPUTATIONAL STEPS

Step 1: Determine O-D Demands and Movement Demands

Either O-D demands or intersection turning movements for the study interchange may be available to the analyst. 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., has only one intersection), the O-D demands and the turning movement demands are the same, and the analysis proceeds similarly to the methodology of Chapter 19, Signalized Intersections, to estimate capacity, v/c, delay, and queue storage ratios. The Applications section of Part B provides guidance on converting O-D movements to turning movements and vice versa for each type of interchange configuration addressed in this methodology.

Step 2: Determine Lane Groups

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 as described in Chapter 19, Signalized Intersections.

Step 3: Determine Adjusted Saturation Flow Rates

The saturation flow rate for each lane group can be measured in the field or estimated with the following equation:

$$s = s_0 N f_w f_{HVg} f_p f_{bb} f_a f_{RT} f_{LT} f_{Lpb} f_{Rpb} f_v f_{LU} f_{DDI}$$

Equation 23-14

where

s = adjusted saturation flow rate (veh/h),

s_0 = base saturation flow rate per lane (from Exhibit 23-21),

N = number of lanes in the lane group,

f_w = adjustment factor for lane width (from Chapter 19),

f_{HVg} = adjustment factor for heavy vehicles and grade (from Chapter 19),

f_p = adjustment factor for existence of a parking lane and parking activity adjacent to the lane group (from Chapter 19),

f_{bb} = adjustment factor for local bus blockage (from Chapter 19),

f_a = adjustment factor for the area type (from Chapter 19),

f_{RT} = adjustment for right-turning vehicle presence in the lane group (from Chapter 19, incorporating f_R interchange saturation flow adjustment No. 4 from Equation 23-19),

f_{LT} = adjustment for left-turning vehicle presence in the lane group (from Chapter 19, incorporating f_R interchange saturation flow adjustment No. 4 from Equation 23-19),

f_{Lpb} = pedestrian adjustment factor for left turns (from Chapter 19),

f_{Rpb} = pedestrian–bicycle adjustment factor for right turns (from Chapter 19),

f_v = adjustment for traffic pressure (interchange saturation flow rate adjustment No. 1 from Equation 23-15),

f_{LU} = adjustment factor for lane utilization (interchange saturation flow rate adjustment No. 2 from Equation 23-16 and Equation 23-17), and

f_{DDI} = adjustment for DDI crossover [interchange saturation flow rate adjustment No. 3 [= 0.913 according to research (*5*)].

Most of the factors in Equation 23-14 are obtained from Chapter 19, Signalized Intersections. The last three, which are not obtained from Chapter 19, are described in greater detail below. A fourth adjustment factor, f_R, which quantifies the effect of turn radius on saturation flow rate of a left- or right-turn movement, is used to modify protected turn movement adjustment factors (provided in Chapter 19) for interchanges. The term f_R is not shown explicitly in the equation, since the radius adjustment modifies the existing f_{LT} and f_{RT} adjustments already in the equation. A different radius adjustment is needed for interchanges, since right- and left-turn radii are often much larger than at a standard intersection, resulting in higher turning speeds and thus a lower impact on saturation flow rates. These four adjustment factors are discussed below.

Interchange Saturation Flow Adjustment No. 1: Traffic Pressure, f_v

The saturation flow adjustment factor for traffic pressure is unique to the interchange methodology as found in research but has not been documented for a standard signalized intersection analysis in Chapter 19.

Saturation flow rates have generally been found to be higher during peak traffic demand periods than during off-peak periods (*9*). Traffic pressure reflects the display of aggressive driving behavior for a large number of drivers during high-demand traffic conditions. Under such conditions, many drivers accept shorter headways during queue discharge than they would under different circumstances. The effect of traffic pressure varies by traffic movement. 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:

Equation 23-15

$$f_v = \begin{cases} \dfrac{1}{1.07 - 0.00672 \times \min(v_i', 30)} & \text{(left turn)} \\ \dfrac{1}{1.07 - 0.00486 \times \min(v_i', 30)} & \text{(through or right turn)} \end{cases}$$

where f_v is the adjustment factor for traffic pressure and v_i' is the 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 23-23 tabulates the results of Equation 23-15 for various demands and for each turning movement type.

Exhibit 23-23
Adjustment Factor for Traffic Pressure (f_v)

Demand Flow Rate $v_{\square}$ (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
30.0 or greater	1.152	1.082

When the lane group is shared by several movements, the adjustment factor for traffic pressure is estimated as the average (weighted on the basis of flows) of the respective movements.

Interchange Saturation Flow Adjustment No. 2: Lane Utilization, f_{LU}

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 such movements may occur at conventional intersections as well, the short links and typically high volumes at interchanges generally result in greater variation in lane distribution. Consideration of ramp configuration makes calculation of lane utilization for O-D pairs more accurate. Segregation at the upstream intersection may occur by driver selection or by designated signing and pavement marking.

To account for these phenomena, lane utilization models have been developed specifically for the external through approaches (surface streets) of two-intersection diamond interchanges, as well as DDIs. The lane utilization factors for all other interchange approaches (freeway ramps, internal approaches, and SPUI approaches) are estimated by using the procedures of Chapter 19. These lane utilization factors are then used to adjust the saturation flow rates for each lane group.

Lane Utilization Adjustment for Diamond Interchanges

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. It is based on the flow in the lane with the highest volume and is calculated by Equation 23-16:

$$f_{LU} = \frac{1}{\%V_{Lmax} \times N}$$

Equation 23-16

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 either on field data or on values obtained from Exhibit 19-15. Equation 23-17 provides a model to estimate the percent of total volume per lane. Exhibit 23-24 provides parameters for each type of interchange configuration and for two-, three-, and four-lane arterials.

Equation 23-17

$$\%V_{Li} = \frac{1}{n} + a_1\left(\frac{v_R}{v_L + v_R + v_T}\right) + a_2\left(\frac{v_L}{v_L + v_R + v_T}\right) + a_3\left(\frac{D \times v_L}{10^6}\right)$$

where

- $\%V_{Li}$ = percent of traffic present in lane L_i, with L_1 representing the leftmost lane, L_2 representing the second lane from the left, and so forth;
- n = number of lanes in the lane group;
- a_i = coefficient for $i = 1$ through $i = 3$ (see Exhibit 23-24);
- D = distance (ft) between the two intersections of the interchange (Equation 23-17 is valid for values of D below 800 ft);
- v_R = O-D demand flow rate traveling through the first intersection and turning right at the second ($v_R = 0$ if there is an exclusive right-turn lane on the external approach);
- v_L = O-D demand flow rate traveling through the first intersection and turning left at the second; and
- v_T = O-D demand flow rate traveling through the first intersection and through the second.

These models estimate the percent of traffic expected to use each through lane as a function of O-D demands on the subject approach. They focus on the external arterial approaches and predict the percent of traffic expected to use a particular lane as a function of downstream turning movements. The turning movements are expressed in terms of their respective O-D flows. O-D Flows A through N are shown in Chapter 34 for each configuration type. When the eastbound and westbound approach patterns are symmetrical, 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.

Exhibit 23-24
Parameters for Lane Utilization Models for the External Arterial Approaches of Diamond and Parclo Interchanges

Interchange Type	Number of Lanes in Lane Group	Leftmost Lane (L_1) a_1	a_2	a_3	Rightmost Lane (L_n) a_1	a_2	a_3
Diamond	2	-0.154	0.187	-0.181	-	-	-
	3	-0.245	0.465	0	0.609	-0.326	0
	4	-0.328	0.684	0	0.640	-0.233	0
Parclo A-2Q	2	0	-0.527	0	-	-	-
	3	0	-0.363	0	0	0.605	0
	4	0	-0.257	0	0	0.747	0
Parclo B-2Q,B-4Q, AB-4Q (WB)	2	0.387	-0.344	0	-	-	-
	3	0.559	-0.218	0	-0.429	0.695	0
	4	0.643	-0.103	0	-0.359	0.794	0
Parclo A-4Q, AB-2Q (EB), AB-4Q (EB)	2	-0.306	-0.484	0	-	-	-
	3	-0.333	-0.289	0	0.579	0.428	0
	4	-0.233	-0.237	0	0.703	0.641	0
Parclo AB-2Q (WB)	2	0.468	0	0	-	-	-
	3	0.735	0	0	-0.308	0	0
	4	0.768	0	0	-0.202	0	0

Notes: If there is an exclusive right-turn lane on the external approach, the O-D demand (v_F or v_G from Exhibit 23-20) should be zero in the respective equation.
Lane utilization of the middle lane (if present) is estimated by subtraction.
In applying these parameters, the highest value is used in the equation.
Refer to Exhibit 23-17 for types of parclo interchanges.

When an external approach has an exclusive right-turning lane, the O-D for that movement (v_F or v_G from Exhibit 23-20) should be assumed to be zero in the respective equation. When there is an additional approach in the upstream intersection, the analyst should use the lane utilization factors of Chapter 19. Equation 23-17 is valid for values of D less than 800 ft. The empirical models underlying Equation 23-17 did not consider configurations with longer distances; for these longer distances between the two intersections, vehicles tend not to pre-position themselves in anticipation of a downstream turn. In those cases, and in the absence of field data, use of the default values of Exhibit 19-15 is recommended. If 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).

Research (5) has shown that as operations approach congested conditions, the lane utilization factor tends to approach 1 (i.e., traffic becomes more uniformly distributed). Lane volume distributions observed in the field should be used if available because they are highly dependent on existing land uses and on 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.

Lane Utilization Adjustment for DDIs

Research at DDIs (*10*) has indicated imbalances in lane utilization similar to those described for other interchanges above. This section presents lane utilization adjustments for the external DDI crossover as a function of left-turn demand ratio at the internal crossover. In addition, the models distinguish between exclusive and shared lane configurations, for a total of five DDI lane configurations shown in Exhibit 23-25. The lane utilization at DDIs is estimated by using Equation 23-18, with parameters for the equation shown in Exhibit 23-26. Only the highest-volume lane is needed to calculate the lane utilization factor. Depending on the availability of data, the model can then be categorized into regimes. The lane with the highest lane utilization ratios is chosen as the representative model for the intersection and is indicated in the table. For design purposes, knowledge of queue lengths per lane is critical in ensuring that adequate storage is provided at the DDI approach.

Exhibit 23-25
Five Categories for DDI Lane Utilization

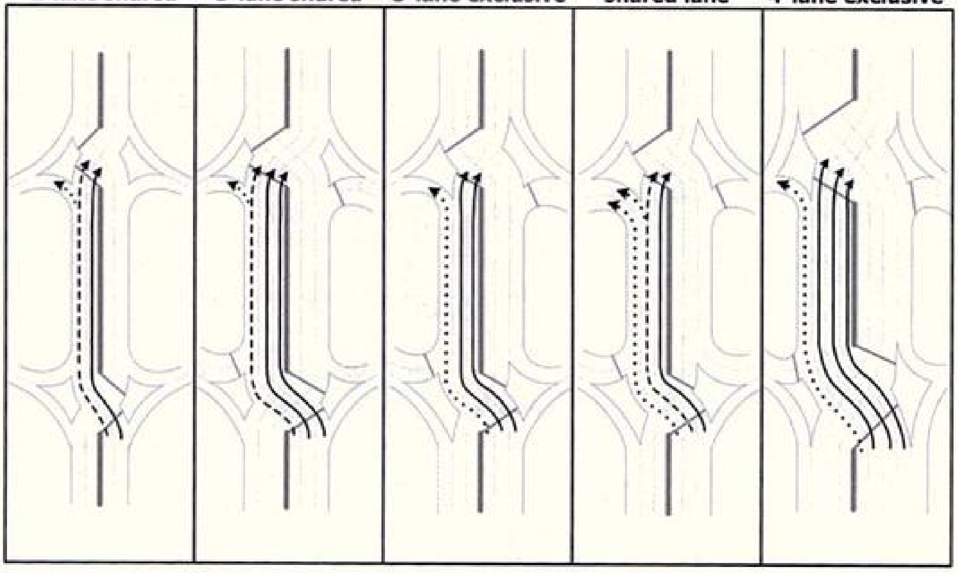


Exhibit 23-26
Lane Utilization Model Coefficients for DDIs

Lane Configuration	Regime	Lane	a_1	a_2
2-lane shared	I (LTDR ≤ 0.35)	Left	0.2129	0.5250
	II (LTDR > 0.35)	Left	0.5386	0.4110
3-lane shared	I-1 (LTDR ≤ 0.13)	Middle	-0.1831	0.3863
	I-2 (0.13 < LTDR ≤ 0.43)	Leftmost	0.2245	0.3336
	II (LTDR > 0.43)	Leftmost	0.6460	0.1523
3-lane exclusive	I (LTDR ≤ 0.33)	Middle	-0.5983	0.5237
	II (LTDR > 0.33)	Leftmost	0.9695	0.0096
3-lane exclusive with middle shared lane	I (LTDR ≤ 0.50)	Middle	-0.2884	0.5626
	II (LTDR > 0.50)	Leftmost	0.4903	0.1761
4-lane exclusive	I (LTDR ≤ 0.35)	Center-left	-0.5432	0.5095
	II (LTDR > 0.35)	Leftmost	0.9286	-0.0071

Note: LTDR = left-turn demand ratio.

$$\%V_{Li,DDI} = a_1 \times LTDR + a_2$$

Equation 23-18

where

$\%V_{Li,DDI}$ = percent of traffic present in lane L_i for a DDI, with L_1 representing the leftmost lane, L_2 representing the second lane from the left, and so forth;

a_i = coefficient for $i = 1$ and $i = 2$ (see Exhibit 23-26); and

$LTDR$ = left-turn demand ratio (decimal), calculated as left-turn demand at external crossover divided by total approach volume.

The maximum $\%V_{Li,DDI}$ value is used as $\%V_{Lmax}$ in Equation 23-16 to calculate f_{LU} for the DDI.

Interchange Saturation Flow Adjustment No. 3: DDI Factor, f_{DDI}

Research suggests that the DDI saturation flow rate is, on average, 8.7% lower than what is estimated for conventional interchanges, but with considerable variation in that estimate.

Research on DDIs (5) suggests that the conventional diamond interchange model overestimates the saturation flow rate at DDIs. Data collected at approaches to 11 DDIs showed that the average field-measured saturation flows were lower by a factor of 0.913, with a standard deviation of 0.55. The DDI adjustment was estimated after all remaining terms in the saturation flow rate equation were controlled for. This factor suggests that the DDI saturation flow rate is, on average, 8.7% lower than what is estimated for conventional interchanges, but with considerable variation in that estimate.

Interchange Saturation Flow Adjustment No. 4: Turn Radius Effects on Left- or Right-Turning Movements, f_{LT} and f_{RT}

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 affect 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:

$$f_R = \frac{1}{1 + \frac{5.61}{R}}$$

Equation 23-19

where R is the radius of curvature of the left- or right-turning path (at the center of the path), in feet.

For protected, exclusive left-turn lanes,

$$f_{LT} = f_R$$

Equation 23-20

For protected, shared left-turn lanes: Factors for protected turn movements are provided in Chapter 19. The revised left- and right-turn adjustment factors are calculated as follows, as a function of the adjustment factor to account for the effects of travel path radius f_R, the proportion of left-turning traffic P_{LT}, and the proportion of right-turning traffic P_{RT}.

$$f_{LT} = \frac{1}{1 + P_{LT}\left(\frac{1}{f_R} - 1\right)}$$

Equation 23-21

where

f_{LT} = saturation flow adjustment factor for left turns;

P_{LT} = percentage of left turns in lane group; and

f_R = interchange saturation flow adjustment No. 4: turn radius from Equation 23-19.

For protected, exclusive right-turn lanes,

Equation 23-22

$$f_{RT} = f_R$$

For protected, shared right-turn lanes,

Equation 23-23

$$f_{RT} \frac{1}{1 + P_{RT}\left(\frac{1}{f_R} - 1\right)}$$

where

f_{RT} = saturation flow adjustment factor for right turns,

P_{RT} = percentage of right turns in lane group, and

f_R = turn radius from Equation 23-19.

Exhibit 23-27 tabulates the adjustment factor for turn radius for several radii. When the lane group is shared by several movements, the adjustment factor for turn radii is estimated as the average (weighted on the basis of flows) of the respective movements. The adjustment factors for permissive phasing are estimated by using the procedures of Chapter 19.

Exhibit 23-27
Adjustment Factor for Turn Radius (f_R)

	Movement Type	
Radius of the Travel Path (ft)	**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

Step 4: Determine Effective Green Adjustment due to Interchange Operations

The effective green adjustment involves three components: (*a*) adjustment in the effective green of the upstream (external) approaches due to the presence of a downstream queue, (*b*) adjustment in the effective green of the downstream (internal) approaches due to demand starvation, and (*c*) adjustment in the effective green for signalized DDI ramp movements due to overlap phasing.

The adjusted lost time t_L' for external arterial approaches and ramp approaches is estimated as shown below. Input variables for the equations are derived mathematically later:

Equation 23-24

$$t_L' = l_1 + L_{D\text{-}A} + Y - e \qquad \text{(arterial)}$$

Equation 23-25

$$t_L' = l_1 + L_{D\text{-}R} + L_{OL\text{-}DDI} + Y - e \qquad \text{(ramp)}$$

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 (lost time adjustment No. 1) (s),

$L_{D\text{-}R}$ = external ramp lost time due to presence of downstream queue (lost time adjustment No. 1) (s),

$L_{OL\text{-}DDI}$ = lost time on signalized external ramp approach at a DDI due to overlap phasing (lost time adjustment No. 2) (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 23-26

where L_{DS} (lost time adjustment No. 3) 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 23-27

where g' is the effective green time adjusted by presence of a downstream queue (s), G is the green time (s), and t'_L is 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:

$$g' = G + Y - t''_L$$

Equation 23-28

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).

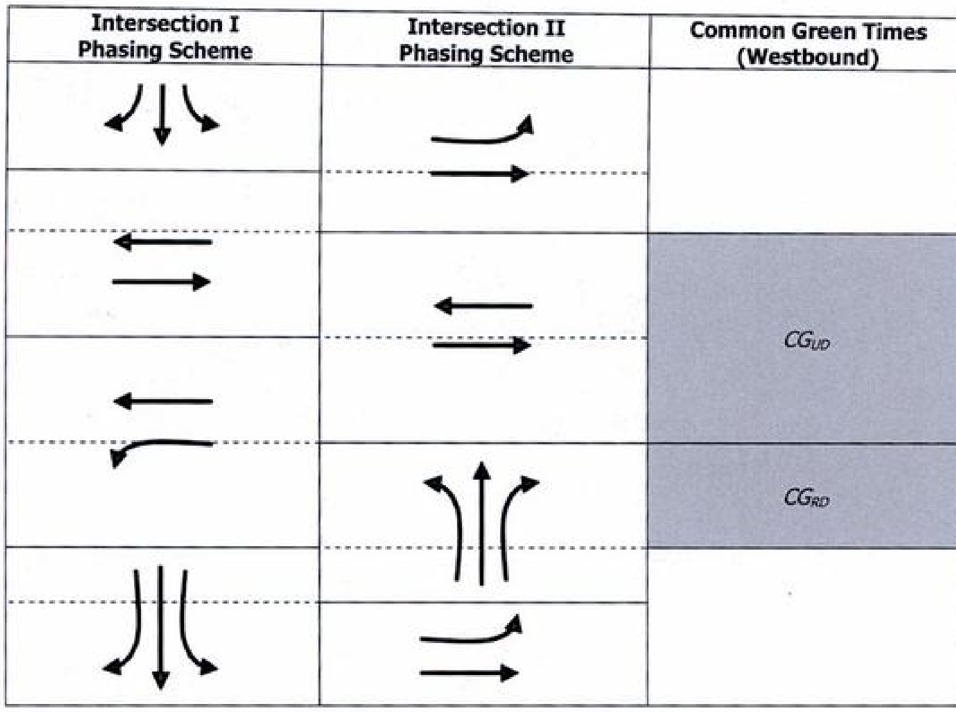
Estimation methods for the additional lost time due to the presence of a downstream link queue and due to demand starvation are given in the following section.

Lost Time Adjustment No. 1 for Diamond Interchanges: Presence of a Downstream Internal Link Queue on Arterial and Ramp, $L_{D\text{-}A}$, $L_{D\text{-}R}$

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.

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 23-28 illustrates an 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 23-28
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:

Equation 23-29

$$L_{D\text{-}A} = G_A - 0.106DQ_A - 5.39\frac{CG_{UD}}{C}$$

Additional lost time on the ramp approach:

Equation 23-30

$$L_{D\text{-}R} = G_R - 0.106DQ_R - 5.39\frac{CG_{RD}}{C}$$

where

$L_{D\text{-}A}$ = lost time on external arterial approach due to presence of downstream queue (s) (min = 0),

$L_{D\text{-}R}$ = lost time on external ramp approach due to presence of downstream queue (s) (min = 0),

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 23-29 or Equation 23-30 results in negative values, 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, the lost time will be zero.

DQ_A and DQ_R are calculated as follows:

$$DQ_A = D - Q_A$$

Equation 23-31

$$DQ_R = D - Q_R$$

Equation 23-32

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 estimated with the following equations:

Queue at the beginning of the upstream arterial Phase A:

$$Q_A = \left(0.0107\frac{v_R}{N_R} - 7.96\frac{G_D}{C} - 0.082CG_{UD} + 7.96\frac{G_R}{C}\right)L_h$$

Equation 23-33

Queue at the beginning of the upstream ramp Phase R:

$$Q_R = \left(0.0107\frac{v_A}{N_A} - 7.96\frac{G_D}{C} - 0.082CG_{RD} + 7.96\frac{G_A}{C}\right)L_h$$

Equation 23-34

where

Q_A = queue at the beginning of upstream arterial Phase A (ft) (min = 0);

Q_R = queue at the beginning of upstream ramp Phase R (ft) (min = 0);

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 subject queue;

N_A = number of arterial lanes feeding 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 a diamond interchange, v_R is the left-turning flow from the ramp, and the variable becomes $v_{Ramp\text{-}L}$. 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, 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 time $L_{D\text{-}A}$ or $L_{D\text{-}R}$ is estimated to be negative, 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.

Lost Time Adjustment No. 1 for DDIs: Presence of a Downstream Internal Link Queue, $L_{D\text{-}A}$, $L_{D\text{-}R}$

In coordinating the two closely spaced DDI signals, a signal designer is able to progress arterial through traffic or favor off-ramp left-turns. Because of the difference in signal phasing, the lost time adjustment for downstream link queues at standard diamond interchanges cannot be readily applied to a DDI. The designer chooses an option on the basis of predominant flows into and out of the corridor during different times of day. In other words, progression of predominant traffic from upstream through movements allows vehicles to pass the downstream signal without stopping, whereas, in the case of predominant traffic from the off-ramp left-turn movement, most off-ramp vehicles are able to pass the downstream signal without stopping. Thus, additional lost time at the upstream intersection due to the downstream internal link queue presence for DDIs is most likely to be considered for either the arterial *through* or the *off-ramp left-turn* movements based on the progression pattern, but not both.

Input data necessary for estimating the additional lost time at DDIs due to an internal queue are presented in Exhibit 23-29.

Exhibit 23-29
Traffic and Geometric Data for Additional Lost Time at DDIs

Input Data and Units	Potential Data Source
Upstream intersection signal plan (s)	Traffic signal plan
Downstream intersection signal plan (s)	Traffic signal plan
Favoring signal plan (through or off-ramp)	Traffic signal plan
Free-flow speed, S_f (mi/h)	Field measurement or default
Average queue spacing in a stationary queue, L_h (ft/veh)	Field measurement or default of 25 ft/veh
Jam density, K_{jam} (veh/mi)	Derived from L_h
Internal link density for arterial through movements, K_I (veh/mi)	Derived from link length and volume
Downstream left-turn demand ratio (decimal)	Field measurement
Upstream through number of lanes (ln)	Field measurement
Upstream off-ramp left-turn number of lanes (ln)	Field measurement
Downstream through number of lanes (ln)	Field measurement
Upstream through volume in the previous cycle that queued in the downstream intersection (veh/ln)	Field measurement
Upstream off-ramp left-turn volume (veh/ln)	Field measurement
Distance between upstream and downstream intersections, D (ft)	Field measurement
Stopping shock wave speed for arterial through movements due to the downstream queue, $V_{W,stop}$ (ft/s)	Greenshields' model
Starting shock wave speed for arterial through movements due to the downstream queue, $V_{W,start}$ (ft/s)	Greenshields' model

Additional lost time at the upstream intersection due to downstream internal queue presence at a DDI can be estimated by using the seven steps provided below:

(a) Obtain basic traffic and geometric data presented in Exhibit 23-29.

(b) Estimate the internal queue length (by using methods in Chapter 19). Identify the position of the last queued vehicle in the internal link from the upstream off-ramp left turn when the upstream arterial through movements receive a green indication.

(c) Identify the first vehicle arrival of the arterial through movement, keeping the average queue spacing obtained as part of Step (a).

(d) Estimate jam density K_{jam} as the inverse of average queue headway (default is 25 ft per passenger car) and internal link density K_I as a function of link length and vehicle volume.

(e) Estimate the stopping shock wave speed by using Greenshields' model (Equation 23-35) with values determined during Step (a).

$$V_{W,\text{stop}} = \frac{-S_f \times K_I}{K_{\text{jam}}} \times \frac{5{,}280 \text{ ft/mi}}{3{,}600 \text{ s/h}}$$

Equation 23-35

(f) Estimate the starting shock wave speed by using Greenshields' model (Equation 23-36) with values determined during Step (a).

$$V_{W,\text{start}} = -S_f \times \frac{5{,}280 \text{ ft/mi}}{3{,}600 \text{ s/h}}$$

Equation 23-36

(g) Estimate the additional lost time at the upstream intersection ($L_{D\text{-}A}$ or $L_{D\text{-}R}$ as appropriate) by subtracting the stopping shock wave intersection time from the starting shock wave intersection time.

There are some caveats in this analytical approach:

- The same free-flow speed is assumed for all vehicles, when in actuality there is some fluctuation.
- Start-up lost time and acceleration and deceleration rates are not considered in the analytical estimation.
- Shock wave speeds are obtained on the basis of Greenshields' model, and the internal density was calculated on the assumption that arterial through vehicles approach the internal queue at 50% of free-flow speed.
- Cycle lengths at the DDI and adjacent intersections are assumed to be the same in a coordinated system.

Even with the caveats, this analytical approach provides a fairly intuitive estimation of how much green time would be lost at the upstream intersection because of downstream internal queue presence with a two-phase signal. Internal queuing patterns are also likely to be sensitive to signal progression patterns between the two DDI crossovers, which can be configured to progress arterial through movements, left turns from the freeway, or a combination.

Lost Time Adjustment No. 2: Overlap Phasing on Signalized External Ramp Approach at a DDI, $f_{OL\text{-}DDI}$

Most DDIs operate by using a signal with two critical movements at each ramp terminal. When one or more off-ramps are signalized, each crossover through movement is concurrent with either the left- or right-turn off-ramp. However, the off-ramp right- and left-turn movements are often a significant distance from the crossover, which makes intersection clearance times (i.e., all-red) longer. Overlap phasing can allow the opposing *crossover through movement* to start before the concurrent *right- or left-turn maneuver*. This permits efficient crossover operation while vehicles clear the ramp terminals. Exhibit 23-30 illustrates how phase overlaps can be used when left- and right-turn ramp movements from the freeway are signalized. Phase overlaps are represented as A, B, C, and D.

The resulting reduction in effective green time for the DDI ramp movements is accounted for through the $L_{OL\text{-}DDI}$ lost time adjustment factor. The lost times applied to DDI off-ramp movements are calculated on the basis of free-flow speeds S_f, clear-zone widths W, vehicle lengths L, and the space between ramp stop bar and conflict zone D. Distance D can be significant, since most exit-ramp stop bars are set to accommodate appropriate sight distances at DDIs. A graphical representation of these distances is provided in Exhibit 23-31.

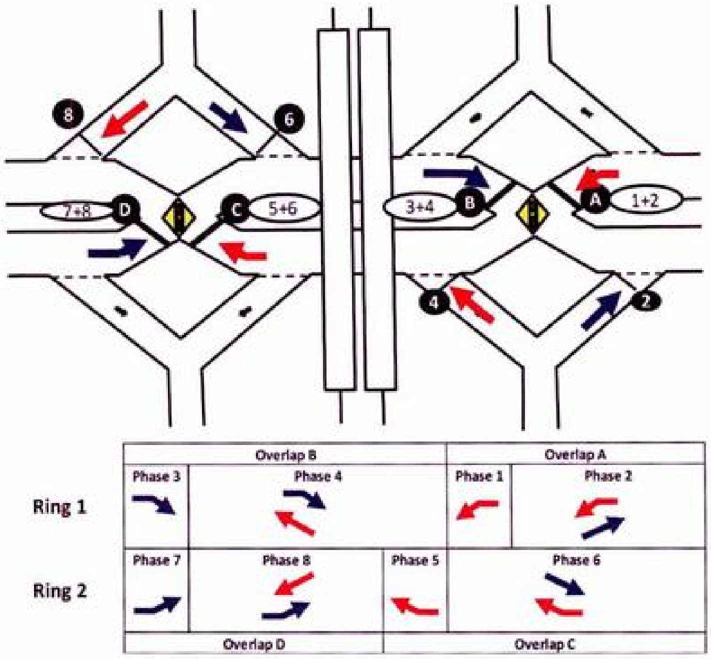


Exhibit 23-30
Standard Phasing Scheme at a DDI with Signalized Ramp Movements

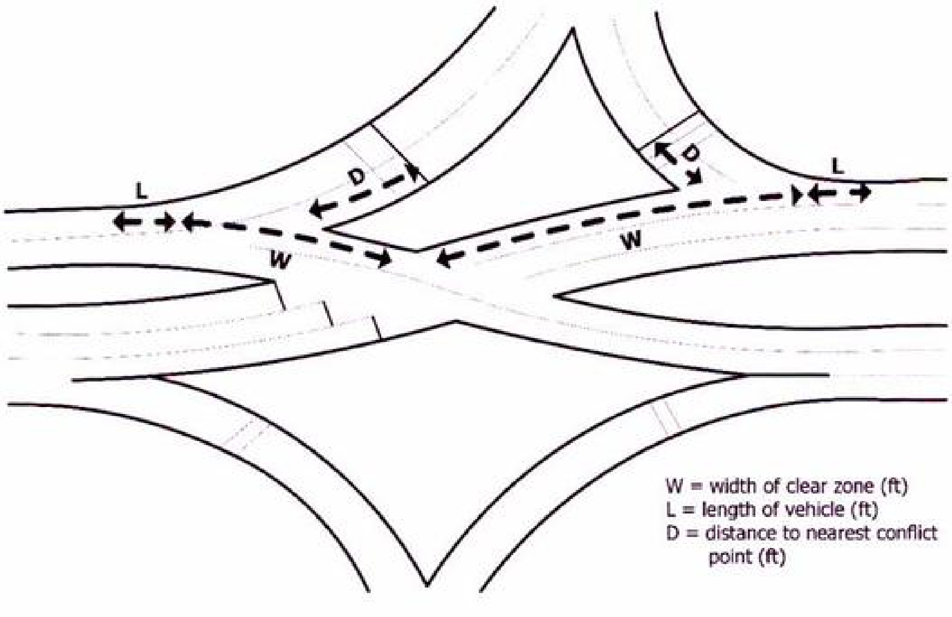


Exhibit 23-31
Graphical Depiction of the Distances Needed to Calculate Off-Ramp Lost Time at DDIs

The overlap phase, required to allow through movements at the crossover to begin before off-ramp left and right turns, can be computed by using the all-red clearance interval calculation. This overlap phase delays the start of a signalized left- or right-turn off-ramp movement (as compared with the concurrent through movement). In essence, this shortens the effective green for signalized ramp terminal movements because of the additional lost time that must be considered. For the purposes of this method, the overlap phase time is included in the analysis as additional lost time for signalized off-ramp movements from the freeway. On the basis of the Exhibit 23-31 variables, lost time for a signalized off-ramp movement can be calculated as follows:

Equation 23-37

$$L_{OL\text{-}DDI} = \frac{W + L - D}{1.467 \times S_f}$$

where

W = width of the clear zone for the longest vehicle path (ft), measured along the centerline of the outside lane, which is the closest conflicting vehicle path to the ramp;

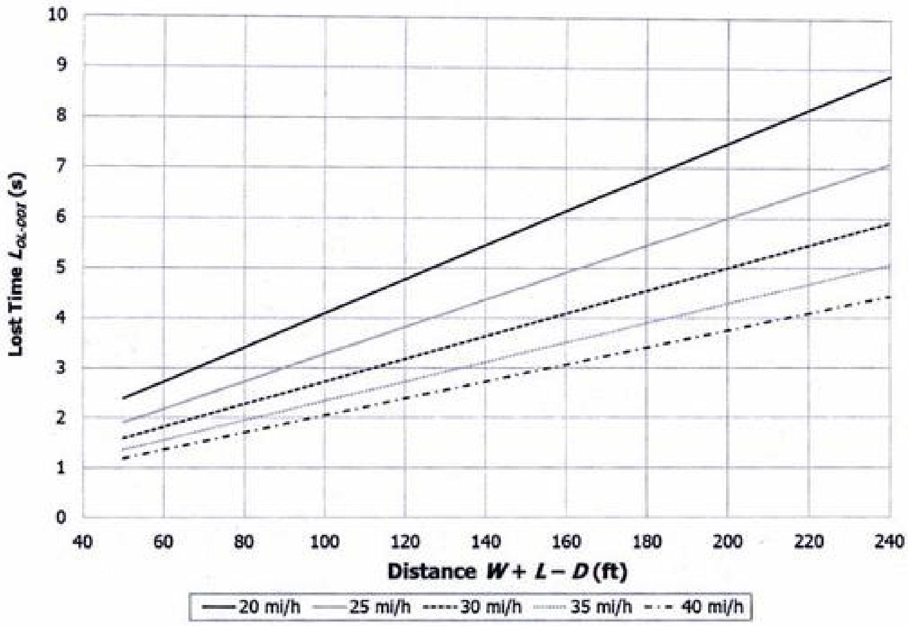
L = design vehicle length (ft), typically 20 ft;

D = distance from the ramp movement stop bar to the conflict point (ft) measured along the centerline of the off-ramp approach; and

S_f = free-flow speed of the vehicle (mi/h).

Exhibit 23-32 provides a graphical representation of Equation 23-37. Speeds ranging from 20 to 40 mi/h are used.

Exhibit 23-32
Lost Time for DDI Off-Ramp Based on Distance Terms



The speed used should provide safe passage of the conflict zone for the vast majority of drivers. If a DDI is newly constructed, the design speed V of the curve should be utilized to provide an estimate of speed; however, to provide safe crossing for the slowest vehicles, it should be decreased by 5 mi/h. Although start-up lost time could be considered for the off-ramp movements, the recommendation is made that it be ignored to allow additional time for conflicting vehicles from the through crossover movement to clear safely as drivers react to the green signal indication at the off-ramp.

Lost Time Adjustment No. 3: Demand Starvation for the Downstream (Internal) Approaches (L_{DS})

This methodology accounts for the effects of demand starvation in diamond 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 23-33 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.

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.

For DDIs, the demand starvation is assumed to be zero. Because of the geometric configuration of DDIs with two directional crossovers and the two-phase signal timing scheme applied at most DDIs, there is generally no opportunity in the cycle for demand starvation to occur. In other words, whenever the outbound movement at the internal crossover has a green indication, one of the upstream movements from the external crossover is being served (either through traffic or left turns from the freeways). For diamond interchanges, the following equation is used to estimate lost time due to demand starvation:

$$L_{DS} = CG_{DS} - Q_{\text{Initial}} \times h_I$$

Equation 23-38

where

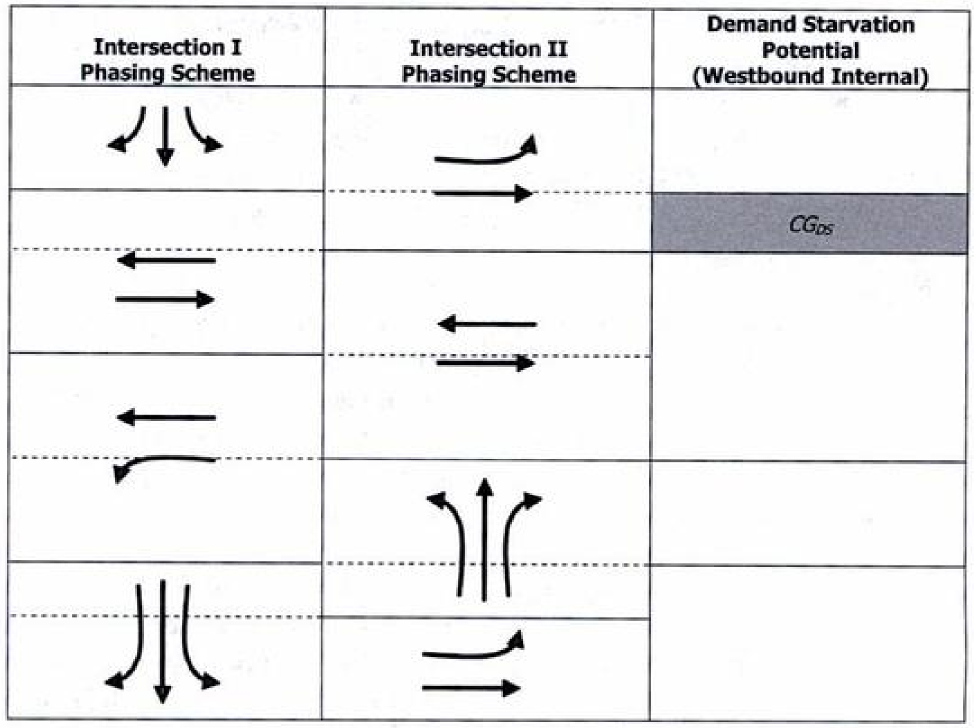
L_{DS} = additional lost time due to demand starvation (s);

CG_{DS} = common green time with demand starvation potential (s), as shown in Exhibit 23-33;

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 23-39.

Exhibit 23-33
Illustration of Interval with Demand Starvation Potential



Equation 23-39

$$Q_{\text{Initial}} = \left[\frac{v_{\text{Ramp-}L} \times C}{N_{\text{Ramp-}L} \times 3{,}600} - \frac{(CG_{RD} - t_L)}{h_I}\right] + \left[\frac{v_{\text{Arterial}} \times C}{N_{\text{Arterial}} \times 3{,}600} - \frac{(CG_{UD} - t_L)}{h_I}\right]$$

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) from Equation 23-24 or Equation 23-25.

Equation 23-38 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 23-39 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 23-39, no vehicles are assumed to 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 that operations at the interchange are not oversaturated.

Step 5: Determine Effective Green Adjustment due to Closely Spaced Adjacent Intersections

The presence of closely spaced signalized intersections in the vicinity of an interchange may affect operations of the entire interchange system and can present unique operational challenges. First, lane utilizations of 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 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 adjacent intersections and have a long-lasting impact throughout the interchange area. Generally, closely spaced signalized intersections whose signals are poorly timed can cause flow blockages on the next upstream link due to queue spillback, even during nominally undersaturated conditions.

The presence of closely spaced signalized intersections in the vicinity of an interchange may affect operations of the entire interchange system and can present unique operational challenges.

In the analysis of interchange ramp terminals, effects of the presence of closely spaced intersections are considered by adjusting lane utilizations of the intersections' arterial approaches, by estimating the additional lost time experienced at the upstream intersection due to the presence of the downstream queue, and by estimating additional lost time due to demand starvation.

Lane utilization factors for through approaches of closely spaced intersections should be estimated by subtracting 0.05 from the lane utilization factors obtained from Exhibit 19-15. Research (*1*) has shown that those 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 23-40

$$L_{D\text{-}U_i} = G_{U_i} - 0.106DQ_i - 5.39\frac{CG_{U_iD}}{C}$$

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 23-31 through Equation 23-34. 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 in Step 4 by Equation 23-38.

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.

Step 6: Determine Performance of YIELD-Controlled Turns

DDIs and other interchanges may feature YIELD-controlled right and left turns from the freeway that need to be considered in the analysis. The procedure presented in this step estimates the delay incurred by YIELD-controlled turns at DDIs, thereby allowing the operations of these movements to be compared with signalized (right- or left-turn) or free-flowing (right-turn) alternatives.

Operations of Free-Flow Turning Movements

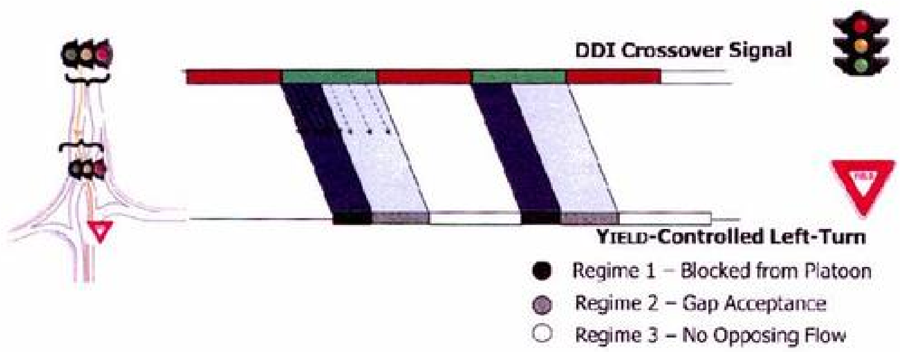
Free-flowing movements are treated the same way as free-flow bypass lanes at roundabouts and signals, with a zero delay for those volumes. Most important, the left-turn movement onto the freeway at the internal signal of a DDI is generally free-flowing (other than pedestrian- and bicycle-induced delay). Some DDIs and other interchange forms also feature free-flowing right turns. These free-flowing volumes are considered to have zero delay, which is included in the interchange's weighted average delay aggregation by approach or movement. The zero delay is also included in the delay and experienced travel time estimate for each O-D movement at the interchange.

Operations of YIELD-Controlled Movements

Some DDIs feature YIELD-controlled left or right turns that need to be evaluated and compared with an alternative of signalizing the turns or providing free-flow lanes (right turns only). YIELD-controlled right turns are also a potential feature of standard diamond interchanges and should be treated consistently. Given the signals upstream of the YIELD-controlled movement, YIELD-controlled left-turn (and right-turn) capacity is evaluated in three flow regimes:

- **Regime 1: blocked by conflicting platoon** when the conflicting signal has just turned green, resulting in zero capacity for the turning movement;
- **Regime 2: gap acceptance in conflicting traffic** after the initial platoon has cleared, with gap acceptance controlled by critical headway, follow-up time, and conflicting flow rate; and
- **Regime 3: no conflicting flow** when the conflicting signal is red, resulting in full capacity, controlled by follow-up time of the YIELD-controlled approach.

Exhibit 23-34
Conflicting Flow Regimes Illustrated for DDI

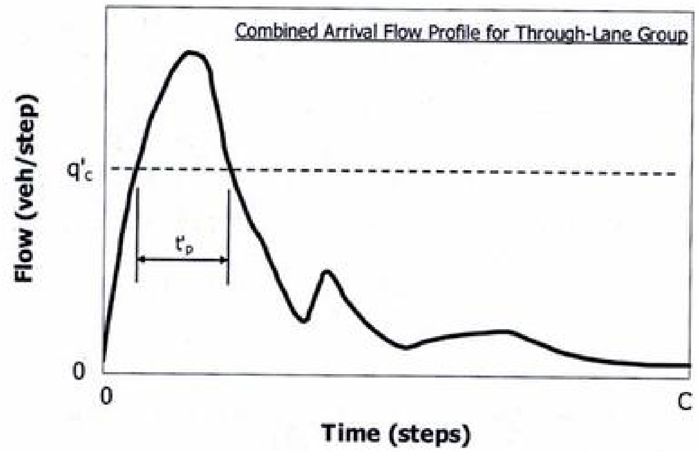


The concept of Regimes 1 and 2 is similar to the procedure for adjacent signal platooning effects on two-way STOP-controlled (TWSC) intersection operations. However, the methodology can be greatly simplified for DDIs, since each turning movement has only one source of conflicting traffic. For a TWSC intersection on an arterial street, platoons occur from four separate movements (the through movement and the left-turn platoon from adjacent signals in two directions). Regime 3 is a new concept, which requires estimation of saturation flow rates for the YIELD-controlled movement without conflicting movements. The three regimes are illustrated in Exhibit 23-34 for a YIELD-controlled left turn.

YIELD-Control Regime 1: Blocked by Conflicting Platoon

The capacity of the YIELD-controlled movement during this regime is zero, since turning vehicles have to yield to the conflicting platoon discharging from the crossover signal (inbound for left turns, outbound for right turns). The method then relies on the estimation of the proportion of time blocked for movement x, which is denoted by $p_{b,x}$. The proportion of time blocked is equal to the amount of time that the conflicting flow rate is high enough to result in headways that are too short to be entered by the YIELD-controlled movement. The critical platoon flow rate q_c is equal to the inverse of the critical headway t_c for that movement (i.e., $q_c = 3{,}600/t_c$), consistent with guidance given in Chapter 30. The appropriate critical headway for DDI turning movements is discussed further below. The resulting blocked period duration t_p' is illustrated in Exhibit 23-35, which is adapted from Exhibit 30-5.

Exhibit 23-35
Estimation of Blocked Period Duration



As a result, the proportion of time blocked for movement x can be estimated explicitly as follows:

Equation 23-41

$$p_{b,x} = \frac{t_p' d_t}{C}$$

where

$p_{b,x}$ = proportion of time blocked for movement x (decimal),

t_p' = blocked period duration (steps),

d_t = time step duration (s/step), and

C = cycle length (s).

One challenge with the formulation above is that it requires an iterative computation of the DDI as part of the time step–based urban street procedure. In a stand-alone DDI evaluation, this factor may be approximated by the time needed to clear the conflicting queue length at the upstream signal, plus the time needed for the last vehicle in the queue to clear the travel distance to the crossover. This method is illustrated in the Extensions to the Methodology section.

YIELD-Control Regime 2: Gap Acceptance in Conflicting Traffic

For the second regime, a gap acceptance–based capacity model is used that mirrors the roundabout capacity procedure. Roundabout approaches and YIELD-controlled turns at DDIs are both YIELD controlled, but gap acceptance behavior needs to be calibrated separately for DDIs because of differences in geometry. The capacity, or maximum entering flow rate, for a roundabout $q_{e,max}$ can be estimated as a function of critical headway t_c, follow-up headway t_f, and the conflicting flow rate by using Siegloch's equation shown below. In this example, $q_{e,max}$ is expressed as the capacity during the gap acceptance regime c_{GA}.

Equation 23-42

$$c_{GA} = \frac{3{,}600}{t_f} \exp\left(-\frac{t_c - \frac{t_f}{2}}{3{,}600} \times q_c\right)$$

where

c_{GA} = capacity during the gap acceptance regime (veh/h),

q_c = conflicting flow rate (veh/h),

t_c = critical headway (s), and

t_f = follow-up headway (s).

Default values for critical headway and follow-up headway were obtained from field data at YIELD-controlled DDIs and are shown in Exhibit 23-36. The gap acceptance–based capacity (Regime 2) for movement x, $p_{GA,x}$, is applied for the duration of the DDI crossover signal green phase that is not blocked by the conflicting platoon, as shown in Equation 23-43.

$$p_{GA,x} = \frac{g - (t_{CQ} + t_{\text{clear}})}{C}$$

Equation 23-43

where

$p_{GA,x}$ = proportion of time of gap acceptance regime (decimal),

t_{CQ} = time to clear conflicting queue (s),

t_{clear} = time for last queued vehicle to clear distance from stop bar to yield point (s),

g = effective green time of the DDI crossover movement (s), and

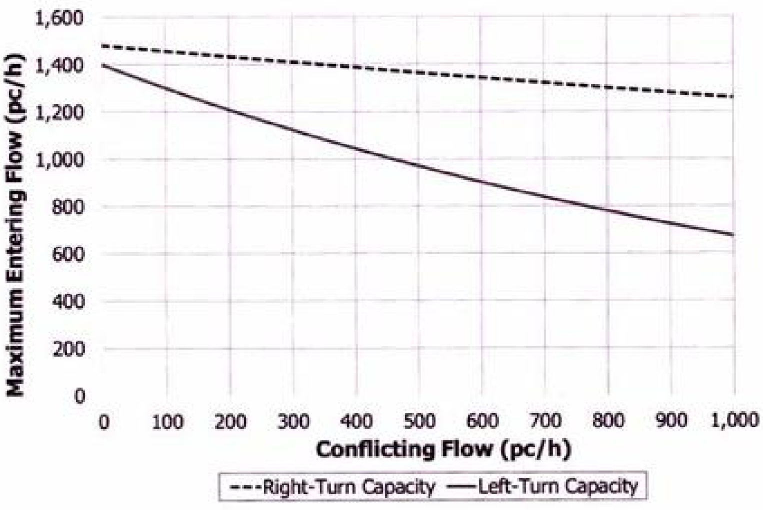
C = cycle length of the DDI crossover signal (s).

Exhibit 23-36
Default DDI Turn Calibration Parameters

Parameter and Units	Left Turns	Right Turns
Critical headway, t_c (s)	3.9	1.8
Follow-up headway, t_f (s)	2.6	2.4
Capacity intercept (veh/h)	1,399	1,481
Capacity slope (veh/h)	-0.00073	-0.00016

The recommended gap acceptance capacity models are represented graphically in Exhibit 23-37. While this discussion is focused on the parameters of Regime 2 (gap acceptance), the intercept of the curves also corresponds to the capacity under the non–conflicting flow condition in Regime 3.

Exhibit 23-37
Plots of Gap Acceptance Capacity Models



YIELD-Control Regime 3: No Conflicting Flow

In Regime 3, the capacity of the YIELD-controlled movement is no longer subject to gap acceptance, since no vehicles can arrive at the yield entry point. The capacity during Regime 3 is controlled by the saturation flow rate of the movement. Given the curved geometry of the approach, this saturation flow rate is expected to be less than that at a standard approach to an intersection.

The saturation flow rate is defined as the inverse of the saturation headway, converted to hours. The capacity of Regime 3 with no conflicting flow, c_{NCF}, is estimated with Equation 23-44.

Equation 23-44

$$c_{NCF} = \frac{3{,}600}{t_f}$$

where

c_{NCF} = capacity of Regime 3 with no conflicting flow rate (veh/h) and

t_f = follow-up headway (s).

The no-conflicting-flow capacity (Regime 3) for movement x, $p_{NCF,x}$, is applied for the duration not used by Regimes 1 and 2. Conceptually, it is equal to the duration of the DDI effective red phase (which is equal to the cycle length minus the effective green time) divided by the cycle length, as shown in Equation 23-45.

Equation 23-45

$$p_{NCF,x} = \frac{r}{C} = \frac{C-g}{C} = 1 - \frac{g}{C}$$

where

$p_{NCF,x}$ = proportion of time of no conflicting flow (decimal),

r = effective red time of the DDI crossover movement (s),

g = effective green time of the DDI crossover movement (s), and

C = cycle length of the DDI crossover signal (s).

Capacity Estimation

The combined capacity of the YIELD-controlled turn, c_{YCT}, can be estimated by the sum of the individual component regime capacities, weighted by the proportion of time each regime is active as shown in Equation 23-46, which can be simplified to Equation 23-47.

Equation 23-46

$$c_{YCT} = c_b \times p_{b,x} + c_{GA} \times p_{GA,x} + c_{NCF} \times p_{NCF,x}$$

Equation 23-47

$$c_{YCT} = \frac{1}{C} \times \left[c_{GA} \times \left(g - t_{CQ} - t_{\text{clear}}\right) + c_{NCF}(C-g)\right]$$

where

c_{YCT} = combined capacity of the YIELD-controlled turn (veh/h);

c_b = capacity during the blocked regime (veh/h), which is zero;

$p_{b,x}$ = proportion of time blocked for isolated DDI analysis (decimal);

c_{GA} = capacity during the gap acceptance regime (veh/h);

$p_{GA,x}$ = proportion of time of gap acceptance regime (decimal);

c_{NCF} = capacity of Regime 3 with no conflicting flow flow rate (veh/h); and

$p_{NCF,x}$ = proportion of time of no conflicting flow (decimal).

The unsignalized movement may be controlled by a STOP sign. The method is not calibrated for such cases. A STOP-controlled approach is expected to result in a reduced capacity relative to a YIELD-controlled approach.

Step 7: Determine *v/c* Ratio and Queue Storage Ratio

The determination of LOS for each O-D involves the calculation of three performance measures: the queue storage ratios R_Q, the v/c ratios, and the average control delays. The queue storage ratios and v/c ratios for each lane group are estimated first. If for any given lane group one or both of these variables exceed 1.0, the LOS for every O-D that travels through that 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 for estimating 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:

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i}$$

Equation 23-48

where

X_i = v/c 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.

Step 8: Determine Control Delay and Experienced Travel Time for Each O-D

This step estimates the average control delay and ETT for each O-D movement. The average control delay for each lane group and movement is estimated by using the procedures provided in Chapter 19, Signalized Intersections. Chapters 20 and 22 provide the delay calculations for YIELD-controlled movements and roundabouts. 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, its average control delay is equal to the average control delay of the respective lane group. If the O-D travels through both intersections, 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 19. 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. ETT is estimated as the sum of intersection control delays d_i and any extra distance travel time (EDTT) due to diverted paths. It is estimated as shown in Equation 23-49:

Equation 23-49

$$ETT = \sum d_i + \sum EDTT$$

The intersection control delay for each junction is estimated by using the control delay procedure in this chapter. For parclo interchanges with loop ramps, EDTT may be estimated from Equation 23-50. For diamond interchanges and DDIs, D_t should reflect the extra distance traveled away from the center of the interchange.

Equation 23-50

$$EDTT = \frac{D_t}{1.47 \times v_D} + a$$

where

$EDTT$ = extra distance travel time (s);

D_t = distance traveled along the loop ramp or diverted movement (ft);

v_D = design speed of the loop ramp or diverted movement (mi/h); and

a = delay due to deceleration into a turn and acceleration after the next turn (s), assumed to be 5 s for a loop ramp movement.

Step 9: Determine LOS

With ETT now determined in Step 8, the LOS for each O-D movement can be estimated by using the thresholds given in Exhibit 23-10. Note that LOS is automatically F, regardless of the ETT value, if the queue storage ratio R_Q for any movement component exceeds 1.0 or if volume exceeds capacity.

4. EXTENSIONS TO THE METHODOLOGY

FINAL DESIGN AND OPERATIONAL ANALYSIS FOR INTERCHANGES WITH ROUNDABOUTS

Roundabouts are generally analyzed with the procedures in Chapter 22. Chapter 34 provides guidance for translating O-D demands into movement demands at a roundabout, in preparation for applying the Chapter 22 procedures.

INTERCHANGES WITH UNSIGNALIZED INTERSECTIONS

Interchanges with unsignalized intersections cannot be evaluated with the procedures of this chapter, since research has not yet been performed on the operation of two closely spaced unsignalized intersections. In the absence of such research, the intersections of such interchanges can be analyzed individually with the procedures of Chapter 20, Two-Way STOP-Controlled Intersections, or Chapter 21, All-Way STOP-Controlled Intersections.

ESTIMATING PROPORTION OF TIME BLOCKED FOR AN ISOLATED DDI

Step 6 in the core interchange procedure for YIELD-controlled turns requires an iterative computation as part of the time step–based urban street procedure to estimate the proportion of time blocked for the YIELD-controlled turning movement. This factor may be approximated in a stand-alone DDI evaluation by the time needed to clear the conflicting queue length at the upstream signal plus the time needed for the last vehicle in the queue to clear the travel distance to the crossover. This is shown in Equation 23-51.

Equation 23-51

$$p'_{b,x} = \frac{t_{CQ} + t_{\text{clear}}}{C}$$

where

$p'_{b,x}$ = proportion of time blocked for *isolated* DDI analysis (decimal),

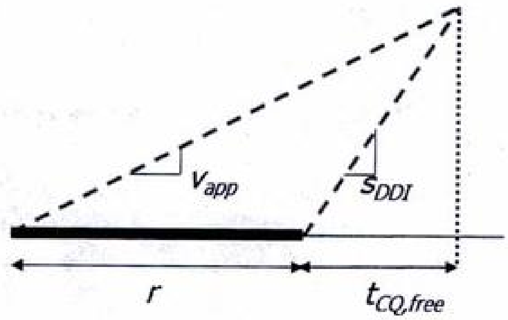
t_{CQ} = time to clear conflicting queue (s),

t_{clear} = time for last queued vehicle to clear distance from stop bar to yield point (s), and

C = cycle length of the DDI crossover signal (s).

For an isolated interchange with assumed random arrivals, the time to clear the conflicting queue $t_{CQ,\text{free}}$ can be approximated from the queuing diagram illustrated in Exhibit 23-38 and calculated from Equation 23-52. This approximation may be overly conservative for DDI crossover signals that do not have random arrivals, since shorter queue lengths often occur in a coordinated signal system. This approximation further assumes that there is no residual queue at the beginning of the red phase resulting from oversaturation in the preceding green interval.

Exhibit 23-38
Queuing Representation as an Approximation of Time to Clear Conflicting Queue for Random Arrivals



Equation 23-52

$$t_{CQ,\text{free}} = \frac{r \times v_{app}}{s_{DDI} - v_{app}}$$

where

$t_{CQ,\text{free}}$ = time to clear conflicting queue for an isolated interchange with random arrivals (s),

v_{app} = approach flow rate (veh/h),

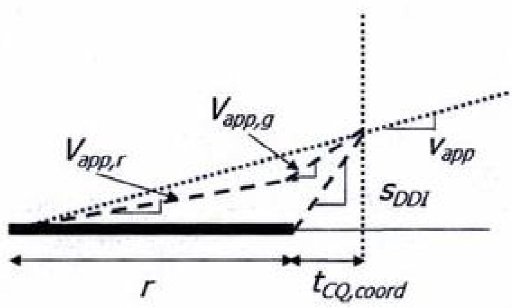
s_{DDI} = saturation flow rate for the DDI approach (veh/h), and

r = duration of the effective red interval for the conflicting movement (s).

For a coordinated interchange (coordinated with external intersections, or coordination between the two DDI crossovers), the assumption of a random arrival distribution is likely not valid. For an interchange in a coordinated system, the time to clear the conflicting queue $t_{CQ,coord}$ can be estimated by considering two distinct arrival types.

Specifically, the method distinguishes between the arrival flow rate during red $v_{app,r}$ and the arrival flow rate during green $v_{app,g}$, as shown in Exhibit 23-39 and calculated in Equation 23-53. This method assumes no residual queue at the beginning of the red phase resulting from oversaturation during the preceding green interval. A comparison between the random and platooned arrivals shows that the conflicting queue length and the time to clear the conflicting queue are much reduced for coordinated arrivals, as illustrated in Exhibit 23-39. Effectively, a reduced time to clear the queue will increase the available time for the gap acceptance regime and will thereby increase the overall capacity of the YIELD-controlled movement.

Exhibit 23-39
Queuing Representation as an Approximation of Time to Clear Conflicting Queue for Coordinated Arrivals



$$t_{CQ,coord} = \frac{C \times (1 - P)}{\frac{S_{DDI}}{v_{app}} - \left[P \times \left(\frac{g}{C}\right)^{-1}\right]}$$

Equation 23-53

where

$t_{CQ,coord}$ = time to clear conflicting queue for a coordinated interchange (s),

P = proportion of arrivals during green (decimal),

v_{app} = approach flow rate (veh/h),

s_{DDI} = saturation flow rate for the DDI approach (veh/h),

g = duration of the effective green interval for the conflicting movement (s), and

C = cycle length of the crossover signal (s).

Note that in the special case when $P = g/c$ (proportion of arrivals equal to g/C), Equation 23-53 simplifies to Equation 23-52. The following three conditions must be met for either Equation 23-52 or Equation 23-53 to apply, to avoid violating the procedure's assumptions:

1. The green flow rate must be less than the saturation flow rate:

   $P \times v_{app} \times (C/g) < S_{DDI}$ or $P < (S_{DDI} \times g)/(v_{app} \times C)$

2. The approach must be undersaturated:

   $v_{app} \times C \leq S_{DDI} \times g$ or $(v_{app}/S_{DDI}) \leq g/C$

3. The time to clear the conflicting queue must be less than or equal to the effective green time:

   $t_{CQ,coord} \leq g$

If any of these assumptions is not met, the interchange should be evaluated as part of an urban street facility analysis by using a computational engine or should be evaluated with alternative (simulation-based) tools.

The time for the last queued vehicle to clear the distance between the stop bar and the yield conflict point t_{clear} is estimated from Equation 23-54, on the assumption that the vehicle was able to reach free-flow speed by the time it reached the stop bar.

$$t_{\text{clear}} = \frac{x_{\text{clear}}}{1.47 \times S_{f,DDI}}$$

Equation 23-54

where

t_{clear} = time for the last queued vehicle to clear the distance between the stop bar and the yield conflict point (s),

x_{clear} = distance between the DDI crossover stop bar and the yield conflict point (ft), and

$S_{f,DDI}$ = free-flow speed between the DDI crossover stop bar and the yield conflict point (mi/h).

Both $p_{b,x}$ and $p'_{b,x}$ were used in Step 6 in the procedure to calculate the overall capacity of the YIELD-controlled movement. The capacity for the blocked period (Regime 1) is assumed to be zero ($c_b = 0$).

This method for approximating the blocked period duration is only needed in a stand-alone DDI evaluation or in the absence of a computational engine. If the DDI is evaluated with a computational engine and integrated into an urban street facility analysis, the calculations are automated by using the concept given in the core procedure and associated discussion.

PEDESTRIAN AND BICYCLE ANALYSIS

Most interchanges provide for pedestrian and bicycle movements. Pedestrians can typically travel through the interchange parallel to the arterial to cross the grade-separated freeway facility. At some interchanges, pedestrian crossings are provided across the arterial; at others, the arterial can only be crossed at adjacent intersections. Bicyclists may choose to use the roadway or (where permitted) the sidewalk network.

No specific methodologies for pedestrian and bicycle LOS for interchanges have been developed to date. However, pedestrian control delay at signalized crossings can be evaluated with methods given in Chapter 19, Signalized Intersections, while pedestrian crossing delay at unsignalized locations may be approximated with the procedure in Chapter 20, Two-Way STOP-Controlled Intersections.

The quality of service of pedestrians and bicyclists traveling along an arterial through an interchange may be approximated with the procedure in Chapter 16, Urban Street Facilities. Because no specific research on pedestrian and bicyclist perceptions and quality of service has been conducted to date at interchanges, this analysis should be performed with care.

5. APPLICATIONS

EXAMPLE PROBLEMS

Section 2 of Chapter 34, Interchange Ramp Terminals: Supplemental, provides 11 example problems that demonstrate the computational steps for the interchange methodology. A listing of these example problems is shown in Exhibit 23-40.

Exhibit 23-40
Listing of Interchange Example Problems Contained in Chapter 34

Example Problem	Description	Application
1	Diamond interchange	Operational
2	Parclo A-2Q interchange	Operational
3	Diamond interchange with four-phase signalization and queue spillback	Operational
4	Diamond interchange with demand starvation	Operational
5	Diverging diamond interchange with signalized control	Operational
6	Diverging diamond interchange with YIELD-controlled turns	Operational
7	Single-point urban interchange	Operational
8	Diamond interchange with closely spaced intersections	Operational
9	Diamond interchange with roundabouts	Operational
10	Compare eight types of signalized interchanges	Interchange type selection
11	Diamond interchange analysis using simulation	Alternative tools

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. The input data include the turning movement demands, number of lanes and their respective lengths and channelization, and traffic control information.

The operational analysis for interchange type selection is found in Chapter 34. It 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.

OPERATIONAL ANALYSIS FOR INTERCHANGE TYPE SELECTION

This type of analysis should be used when the type of interchange is not known yet and the analyst is interested in assessing the traffic operations of various alternatives. Discussion of this approach can be found in Chapter 34, Interchange Ramp Terminals: Supplemental. For this type of analysis, 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 MOVEMENT ESTIMATION

Worksheets in Chapter 34 illustrate how O-D movements can be obtained from turning movements and how turning movements can be obtained from O-D movements, for different interchange types.

USE OF ALTERNATIVE TOOLS

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 on the basis of analysis of individual vehicle trajectories is provided in Chapter 7, Interpreting HCM and Alternative Tool Results, with supplemental examples provided in Chapter 36, Concepts: Supplemental. Chapter 19, Signalized Intersections, provides 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 19, 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 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 Part B of 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 make the production of compatible capacity estimates impossible. 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 19 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 Chapter 34 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. However, situations in which alternative tools might be needed for a proper assessment of performance can be introduced.

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.

Part C: Alternative Intersection Evaluation

1. INTRODUCTION

OVERVIEW

Alternative intersections are created by rerouting one or more movements from their usual places to secondary junctions. Often, the rerouted movements are left turns. Alternative intersection forms such as the jughandle and the median U-turn have been used in New Jersey and Michigan, respectively, for decades. Other alternative intersection designs are newer. Alternative intersections may be used to enhance safety or reduce delay. Previous editions of the HCM provided methods by which the individual pieces of an alternative intersection could be evaluated. However, this part provides a methodology for evaluating an alternative intersection as a whole, including the primary junction, any secondary junctions, and any extra rerouted travel that motorists experience. This methodology can be applied to three of the most common alternative intersection types used in the United States: the restricted crossing U-turn (RCUT), the median U-turn (MUT), and the displaced left turn (DLT).

RCUTs are also known as the J-turn, the reduced-conflict intersection, the superstreet, and the synchronized street.

The MUT is also known as the Michigan left turn or the thru turn.

The DLT is also known as the continuous-flow intersection.

The RCUT maintains all mainline left, through, and right moves with no rerouting. However, an RCUT reroutes the minor-street left turn and through movements to one-way U-turn crossovers on the major street. These crossovers are typically located 450 ft or more from the central junction. Because RCUTs reroute minor-street through movements, they are typically used where minor-street demands are below 25,000 veh/day. However, they could accommodate much higher minor-street demand if the right-turn proportion is particularly high. An MUT maintains all mainline and side street through and right moves with no rerouting. However, it reroutes all left turns to one-way U-turn crossovers typically located on the major street 500 to 800 ft from the central junction. Because MUTs reroute all left turns, they are typically used where left-turn demands are relatively low. The DLT reroutes left turns to crossovers upstream of the central junction; the left-turn traffic streams then approach the central junction to the left of the opposing through movement. DLTs can move left-turn and through vehicles during the same signal phase without conflict, so they are typically used where maximum vehicle capacity is desired.

PART ORGANIZATION

This part is organized into five sections, including this introductory section. The second section provides more detailed descriptions of RCUTs, MUTs, and DLTs and presents the unique operational characteristics of these three designs that must be considered by analysts. The third presents the core analysis methodology. The fourth describes extensions to the core methodology for alternative intersection types not covered by the core methodology and provides guidance for analyzing pedestrian and bicycle operations. The fifth describes potential applications of the methodology, presents example results, and provides guidance on the use of alternative tools.

2. CONCEPTS

RESTRICTED CROSSING U-TURN AND MEDIAN U-TURN INTERSECTIONS

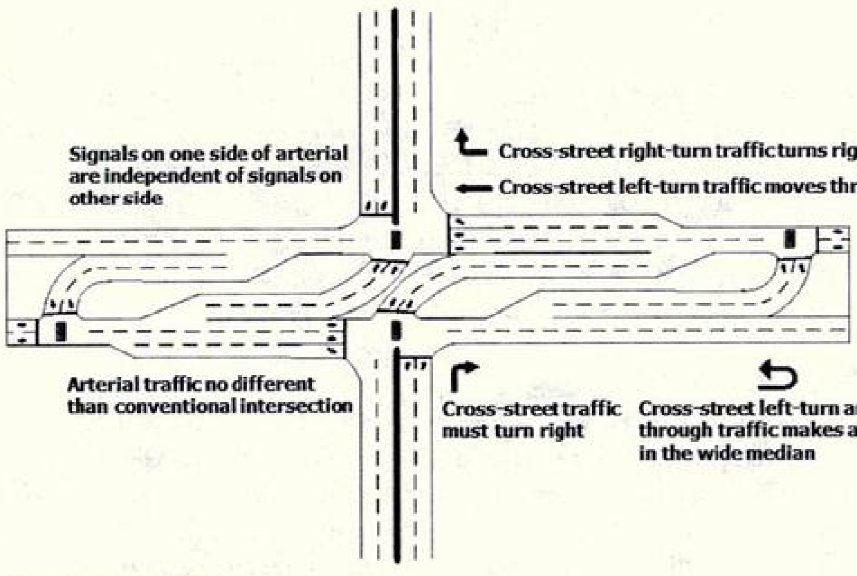
RCUTs and MUTs can be controlled by traffic signals, STOP control on the minor-street approach, or merges and diverges. The core computational methodology in Section 3 can evaluate all three types of control at RCUTs and MUTs with three or four approaches. RCUTs and MUTs with signals are typically built in urban or suburban areas with higher traffic demands, while those with STOP signs or merges are typically built in rural areas on high-speed roadways with lower minor-street traffic demands. RCUTs with signals or STOP signs and MUTs typically have 450 to 800 ft from the main junction to a U-turn crossover. RCUTs with merges typically have more than 800 ft from the main junction to a U-turn crossover, to make the weaving maneuvers easier.

Exhibit 23-41 through Exhibit 23-43 illustrate three types of RCUT designs covered by the methodology, while Exhibit 23-44 illustrates one of the MUT designs covered by the methodology. These illustrations are not to scale, and the number of lanes shown is illustrative. RCUT and MUT intersections can have one- or two-lane crossovers, may or may not have exclusive right-turn lanes, and may have one to four through lanes per approach.

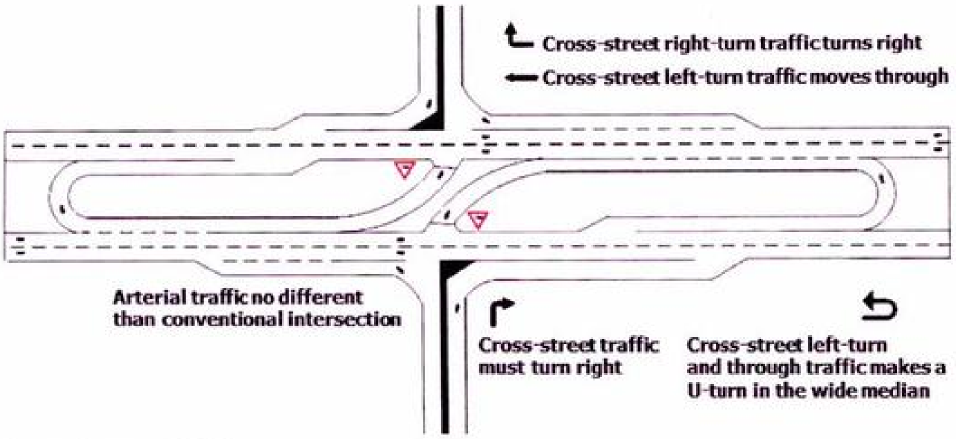
YIELD control is not considered to be significantly different from STOP sign operation for the purpose of this method.

Some agencies use YIELD signs to control minor-street, U-turn crossover, or left-turn crossover operations instead of STOP signs. For purposes of the operational analysis procedure, YIELD-sign operation is not considered to be significantly different from STOP-sign operation. The remainder of this part will simply refer to STOP control.

Exhibit 23-41
Four-Legged RCUT with Signals

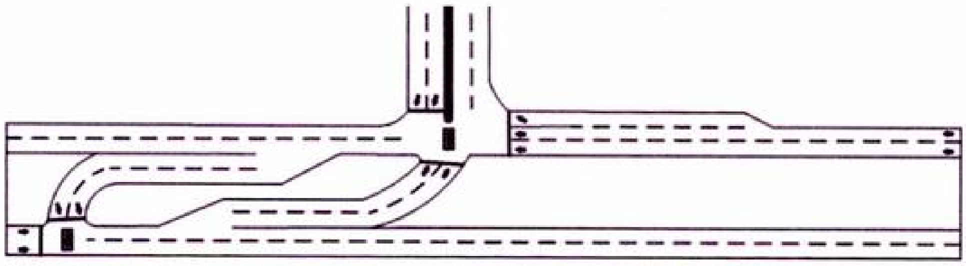


Source: Hummer et al. (*6*).



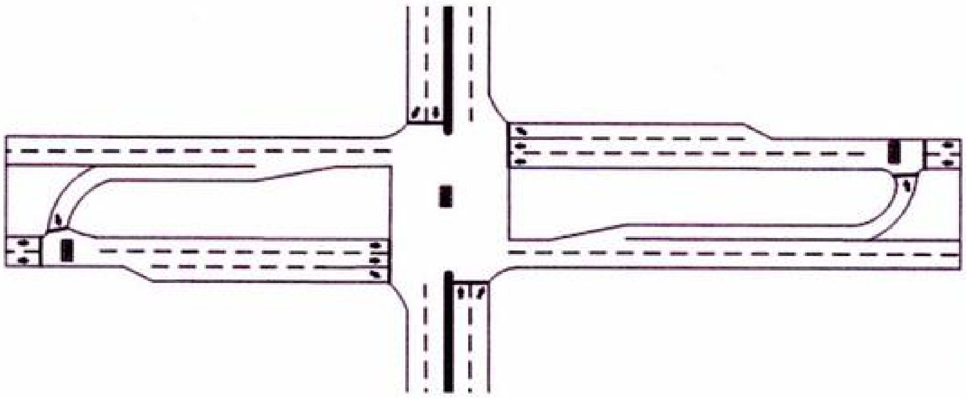
Source: Hummer et al. (*6*).

Exhibit 23-42
Four-Legged RCUT with Merges and Diverges



Source: Hummer et al. (*6*).

Exhibit 23-43
Three-Legged RCUT with Signals



Source: Hummer et al. (*6*).

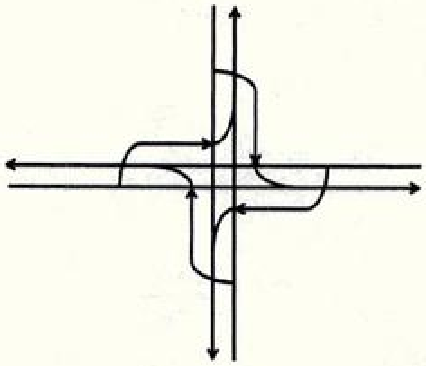
Exhibit 23-44
Four-Legged MUT with Signals

Other types of RCUTs covered in this chapter include three-legged with merges and diverges, three-legged with STOP signs, and four-legged with STOP signs. Two types of four-legged MUTs are covered by this part's methodology. Both have a signalized main junction. One has signalized U-turn crossovers, while the other has STOP-controlled U-turn crossovers. A three-legged MUT with a signal-controlled U-turn crossover is the same design as the three-legged RCUT in Exhibit 23-43. This part's methodology also covers three-legged MUTs with a signal-controlled main junction and a STOP-controlled U-turn crossover.

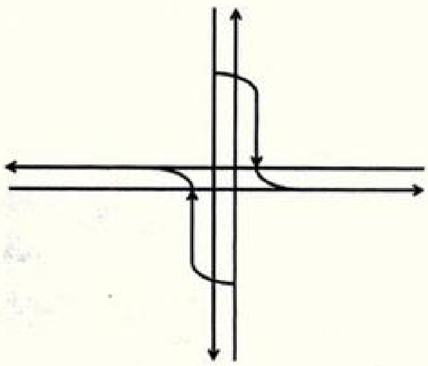
DISPLACED LEFT-TURN INTERSECTIONS

DLT intersections provide one or more left-turn crossover locations several hundred feet upstream of the main intersection. The crossover locations are typically signalized. They may be referred to as "supplemental" intersections, because their purpose is to supplement (improve) the efficiency of the main intersection. At "full" DLT intersections, supplemental intersections are present on all approaches to the main intersection. At "partial" DLT intersections, supplemental intersections exist on some, but not all, approaches to the main intersection. Exhibit 23-45 contrasts the full and partial DLT designs.

Exhibit 23-45
Roadway Geometry for Full and Partial DLT Intersections



(a) Full DLT



(b) Partial DLT

These supplemental intersections, which are typically signal-controlled, are installed to eliminate left-turn conflicts and left-turn phases at the main intersection.

These supplemental intersections, which are typically signal-controlled, are installed to eliminate left-turn conflicts and left-turn phases at the main intersection. Left-turning vehicles cross the opposing through traffic lanes at the supplemental intersection and then approach the cross street on a separate, channelized set of turn lanes on the outside of the opposing travel lanes. On reaching the cross street, left-turning movements are served by the same green phase as opposing through traffic, without the conflicts that exist at a conventional intersection. Exhibit 23-46(a) illustrates a dual left-turn lane crossover at the upstream supplemental intersection that approaches the cross street on a separate road.

Exhibit 23-46(b) shows a right-turn lane from the cross street that is channelized to the outside of the left-turn lanes. The channelized right-turn lane provides three possible benefits: (*a*) right-turning vehicles bypass the main intersection without stopping; (*b*) right-turning vehicles bypass the downstream supplemental intersection without stopping; and (*c*) an advanced design exists in which opposing left-turn vehicles merge into the channelized right-turn lane, to bypass the downstream supplemental intersection without stopping. However, some states have chosen not to build channelized right-turn lanes when right-of-way costs are predicted to exceed the operational benefits.

Exhibit 23-46
Lane Geometry for DLT Intersections

(a) Displaced Left-Turn Roadway (b) Right-Turn Channelization Options

Source: Hughes et al. (*7*).

In many cases, where left-turn and through demand volumes are both sufficiently high, DLT intersections are expected to operate more efficiently than conventional intersections. However, this efficiency may depend on many factors, including relative left-turn and through demands on the approaches, demands on the opposing approaches, geometric design elements such as crossover angle, and overall intersection demand volume. This part's methodology may help in performing a more careful analysis of this intersection type's operational efficiency. Further information about DLT operational characteristics and benefits of the DLT intersection are provided in an FHWA publication (*8*).

3. CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

Spatial and Temporal Limits

Part A of this chapter discusses spatial and temporal limits for alternative intersection analysis.

Performance Measures

The operational analysis methodology for alternative intersections provides the performance measure *experienced travel time* (ETT). For each O-D movement, the ETT includes the control delay experienced at each junction encountered, plus the time experienced to travel any extra distances required by the design. The methodology computes control delays at each individual junction that makes up the alternative intersection, and capacity and v/c ratios are available for each of those junctions. The method also computes LOS for each O-D movement.

Strengths of the Methodology

The strengths of the operational analysis methodology outlined in this section lie in its ability to estimate the performance of alternative intersection forms and to compare their performance with that of conventional intersections and interchanges. The following are other strengths of the methodology: (*a*) it establishes a framework for the analysis of alternative intersections, for which there is a growing need; (*b*) it establishes LOS criteria for evaluation of performance of alternative intersections and comparison with other forms of control and configuration; (*c*) it provides a recommended method for integrating current HCM principles, procedures, and computations into the analysis of alternative intersections; and (*d*) it establishes a basis for further research into the operation of alternative intersections (e.g., arterial weaving).

Limitations of the Methodology

The overall methodology is relatively new and has thus received a limited amount of validation. Analysts should also keep in mind that the two sources of ETT—control delay and extra distance travel time (EDTT)—are weighted equally by the LOS methodology. A different weighting is possible—for example, motorists could weight time spent traveling as being less onerous than time spent in a queue—but there is no definitive research at this point to support providing different weights to different ETT components.

Alternative Tool Considerations

Alternative tools like microsimulation have been applied to alternative intersections for many years and have been validated against field data. Results from simulation may be translated to the LOS framework presented in this chapter with confidence as long as analysts keep certain caveats in mind. First, analysts should be aware that simulation models will have to be calibrated to match some of the recommended default values provided in this procedure, such as the saturation flow at a signalized U-turn crossover or the critical headway at a U-turn crossover with a STOP sign. Second, analysts must ensure that performance measures from the simulation model match the ETT produced by this procedure. If the simulation model provides travel times by link and delay estimates by node, this matching could be done several ways. For example, the analyst could use simulation results to create an estimate of ETT as the sums of correct component delays and travel times. An alternative procedure would be to make simulation runs on networks with no control delay and no extra distance traveled. Travel time results from those runs could then be subtracted from travel time estimates made during runs with the alternative intersection in place.

REQUIRED DATA AND SOURCES

Generally, the same data are required for performing an operational analysis on an alternative intersection as are required for analyzing each individual junction with the methods from Chapter 19, Signalized Intersections, and Chapter 20, Two-Way STOP-Controlled Intersections. A few other inputs are needed to account for the extra travel distances covered by rerouted movements. At the individual junctions, the required data generally include traffic demand data, traffic control device data, and intersection geometry data. For individual junctions with signal control, Exhibit 19-11 and Exhibit 19-12 show the list of required input data; signal coordination is an important feature at DLTs, MUTs with signals at the U-turn crossovers, and signalized RCUTs, so the signal data in Exhibit 19-12 should not be neglected. For individual junctions with two-way STOP control, Exhibit 20-5 shows the list of required input data.

Collecting required input data for the individual junctions is not significantly different from the process at a conventional intersection, with the important exception of the collection of demand flow rates for each movement at an existing RCUT or MUT. At an RCUT or MUT, the distance between main junction and U-turn crossovers is typically too great for both to be observed by a single person or camera. Consequently, the recording of turning movement demands in the field requires more labor or equipment than usual. In addition, manual or camera observations at the main junction of an MUT may require linkage to those at the U-turn crossover to determine, for example, which vehicles turning right from the minor street then used the crossover and were actually left-turning vehicles. Section 2.1.4.2 of the *Manual of Transportation Engineering Studies,* 2nd edition, discusses how to make turning movement counts at RCUTs and MUTs (*11*).

The following are the additional required data to analyze an RCUT, MUT, or DLT:

- Volume of U-turns on red at a signalized crossover,
- Median width at a signalized U-turn crossover,
- Distance from the U-turn crossover (at an RCUT or MUT) or left-turn crossover (at a DLT) to the main junction, and
- Free-flow speed along the major street between crossovers.

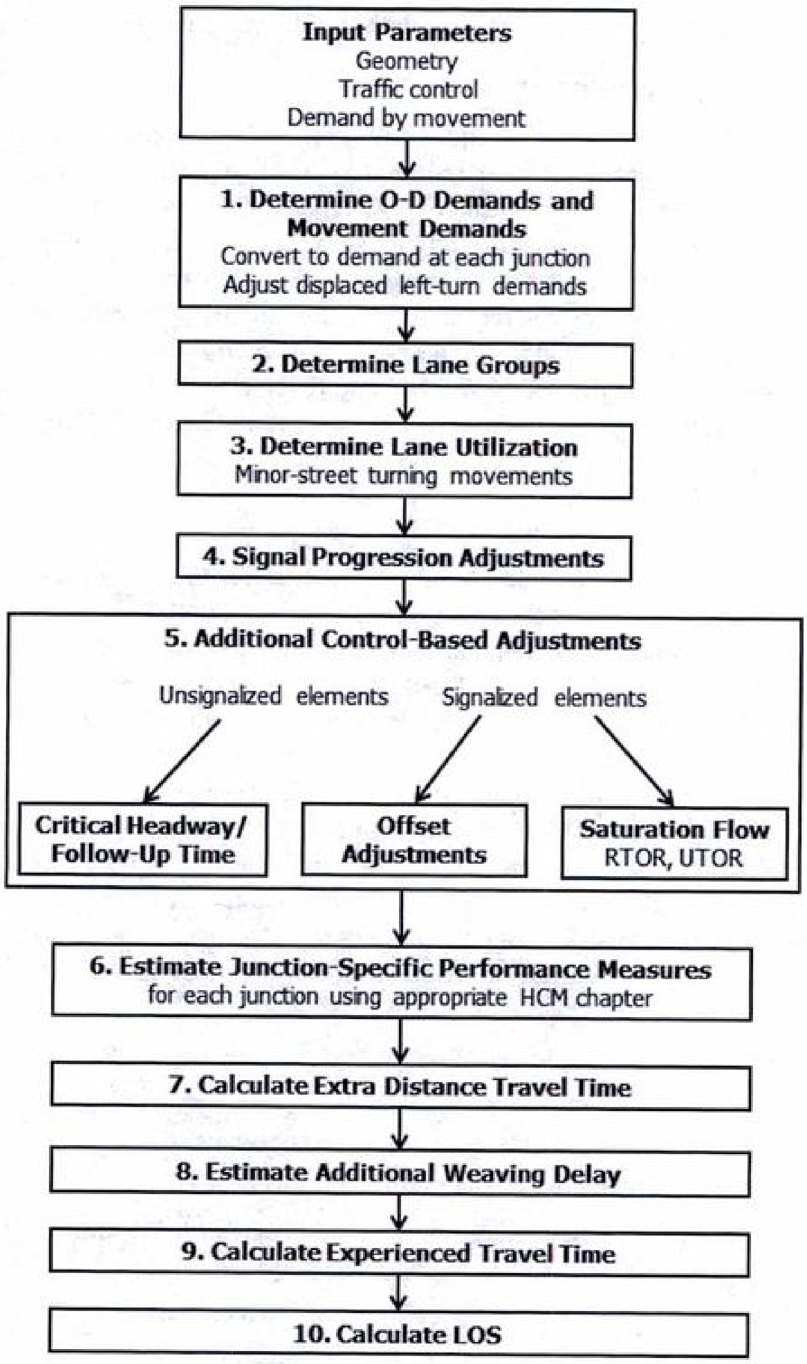
If the free-flow speed along the major street between the crossovers is unavailable, analysts can use the method outlined in Chapter 18, Urban Street

Segments, to estimate that variable. Turning radius data at crossovers are not required by the procedure, because of the availability of default values. However, use of field-measured turning radii is recommended to improve the accuracy of the saturation flow rate estimation.

OVERVIEW OF THE METHODOLOGY

Exhibit 23-47 shows the basic steps for evaluating alternative intersections.

Exhibit 23-47
Operational Analysis Framework for Alternative Intersections



Note: RTOR = right turns on red, UTOR = U-turns on red.

The same basic analysis steps are used for alternative intersections as for conventional intersections. Only Steps 1 and 10 are performed for the intersection as a whole. Steps 2 through 6 are performed for each individual junction in the alternative intersection. Steps 7 and 8 are performed for each relevant link between a main junction and a U-turn crossover at an RCUT or MUT. Step 9 is performed for each movement through the alternative intersection. The subsections below describe the application of each step to each movement at RCUTs, MUTs, and DLTs. Since the procedures for RCUTs and MUTs are similar, they are discussed together despite RCUTs and MUTs being different types of intersections intended to serve different demand patterns.

COMPUTATIONAL STEPS

Restricted Crossing U-Turn and Median U-Turn Intersections

Step 1: Determine O-D Demands and Movement Demands

The standard turning movement demand pattern of left turn, through, and right turn from each approach is converted into left turns, through vehicles, and right turns at each component junction.

Step 2: Determine Lane Groups

Step 2 is performed for each signalized junction within the RCUT or MUT intersection. The determination of lane and movement groups is not relevant for RCUT junctions with STOP signs or merges or for an MUT with STOP signs.

During this step, analysts determine appropriate movement and lane groups and convert forecast or counted volumes into flow rates. 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.

$$v_i = \frac{V_i}{PHF}$$

Equation 23-55

where v_i is the demand flow rate for movement i (veh/h), V_i is the demand volume for movement i (veh/h), and PHF is the peak hour factor.

Analysts should apply the rules described in Steps 2 and 3 of the Chapter 19 methodology to find flow rates per lane and movement group.

Step 3: Determine Lane Utilization

Step 3 estimates the appropriate lane utilization factor for lane groups with multiple lanes. This step is only needed at RCUT and MUT junctions with signals and for lane groups with multiple lanes and at least one shared lane. Approaches to RCUT junctions with STOP signs or merges or MUT junctions with STOP signs typically have just one lane.

RCUT with Multiple-Lane Left-Turn Crossover

Analysts should obtain field data if available. If no field data are available, analysts should use the lane utilization factor default values in Exhibit 19-15.

RCUT or MUT Signalized Dual-Lane U-Turn Crossover

Analysts should use field data if possible, since the factors involved could be complex. If a field study is not possible, an initial estimate of the lane utilization factor should be based on the eventual destination of the traffic stream. Vehicles heading for the major street should be placed in the left lane, and vehicles headed for the minor street should be placed in the right lane. Analysts should apply common sense to this initial lane assignment to ensure that the placements are not too unbalanced.

RCUT or MUT Signalized Multiple-Lane Right Turn

Analysts should use field data if possible, since the factors involved could be complex. If a field study is not possible, the lane utilization factor should be based on the eventual destination of the traffic stream.

For a two-lane minor-street approach at an RCUT, left-turning and through vehicles heading for the U-turn crossover should be placed in the left lane and right-turning vehicles should be placed in the right lane. For a three-lane minor-street approach at an RCUT, left-turning vehicles heading for the left lane of the U-turn crossover should be placed in the left lane, through vehicles heading for the right lane of the U-turn crossover should be placed in the middle lane, and right-turning vehicles should be placed in the right lane.

For a two-lane exclusive right-turn lane on the minor-street approach at an MUT, left-turning vehicles heading for the U-turn crossover should be placed in the left lane and right-turning vehicles should be placed in the right lane. If the minor-street approach to an MUT includes a shared through and right lane plus an exclusive right-turn lane, the lane distribution can be estimated with Step 4 of the core methodology for roundabouts provided in Chapter 22. In all of these cases, analysts should consider modifying these initial distributions if they produce large demand imbalances that do not appear logical.

Step 4: Signal Progression Adjustments

This step assembles the data to account for progression through multiple signals properly. Exhibit 23-48 shows which signals are encountered for each movement through each type of four-legged RCUT analyzed by the methodology. Exhibit 23-49 and Exhibit 23-50 show similar information for each type of three-legged RCUT and each type of MUT, respectively, analyzed by the methodology. There may be non-random arrivals at the initial RCUT signal encountered. Arrivals at the second or third signals encountered will likely have non-random arrivals as well.

Exhibit 23-48
Junctions and Extra Travel Time Segments at a Four-Legged RCUT

Intersection Type	Movement	Traffic Control Device: First	Second	Third	EDTT?
With signals	Left from major street	Through at U-turn crossover signal	Left turn at main junction signal	None	No
	Major-street through	Through at U-turn crossover signal	Through at main junction signal	None	No
	Right from major street	Through at U-turn crossover signal	Right turn at main junction signal	None	No
	Left from minor street	Right turn at main junction signal	U-turn at crossover signal	Through at main junction signal	Yes
	Minor-street through	Right turn at main junction signal	U-turn at crossover signal	Right turn at main junction signal	Yes
	Right from minor street	Right turn at main junction signal	None	None	No
With STOP signs or merges and diverges	Left from major street	Left turn at main junction stop	None	None	No
	Major-street through	None	None	None	No
	Right from major street	None	None	None	No
	Left from minor street	Right turn at main junction stop/merge	U-turn at crossover stop/merge	None	Yes
	Minor-street through	Right turn at main junction stop/merge	U-turn at crossover stop/merge	None	Yes
	Right from minor street	Right turn at main junction stop/merge	None	None	No

Note: EDTT = extra distance travel time to and from U-turn crossover.

Exhibit 23-49
Junctions and Extra Travel Time Segments at a Three-Legged RCUT

Intersection Type	Movement	Traffic Control Device: First	Second	Third	EDTT?
With signals	Left from major street	Through at U-turn crossover signal	Left turn at main junction signal	None	No
	Major-street through (top of T)	Through at U-turn crossover signal	None	None	No
	Major-street through (stem side)	Through at main junction signal	None	None	No
	Right from major street	Right turn at main junction signal	None	None	No
	Left from minor street	Right turn at main junction signal	U-turn at crossover signal	None	Yes
	Right from minor street	Right turn at main junction signal	None	None	No
With STOP signs or merges and diverges	Left from major street	Left turn at main junction stop	None	None	No
	Major-street through (top of T)	None	None	None	No
	Major-street through (stem side)	None	None	None	No
	Right from major street	None	None	None	No
	Left from minor street	Right turn at main junction stop/merge	U-turn at crossover stop/merge	None	Yes
	Right from minor street	Right turn at main junction stop/merge	None	None	No

Note: EDTT = extra distance travel time to and from U-turn crossover.

Exhibit 23-50
Junctions and Extra Travel Time Segments at an MUT

Intersection Type	Movement	Traffic Control Device First	Second	Third	Fourth	EDTT?
Four-legged with signals at U-turn crossovers	Left from major street	Through at U-turn crossover	Through at main junction	U-turn at crossover	Right turn at main junction	Yes
	Major-street through	Through at U-turn crossover	Through at main junction	None	None	No
	Right from major street	Through at U-turn crossover	Right turn at main junction	None	None	No
	Left from minor street	Right turn at main junction	U-turn at crossover	Through at main junction	None	Yes
	Minor-street through	Through at main junction	None	None	None	No
	Right from minor street	Right turn at main junction	None	None	None	No
Four-legged with STOP signs at U-turn crossovers	Left from major street	Through at main junction	U-turn at crossover stop	Right turn at main junction	None	Yes
	Major-street through	Through at main junction	None	None	None	No
	Right from major street	Right turn at main junction	None	None	None	No
	Left from minor street	Right turn at main junction	U-turn at crossover stop	Through at main junction	None	Yes
	Minor-street through	Through at main junction	None	None	None	No
	Right from minor street	Right turn at main junction	None	None	None	No
Three-legged with signal at main junction and STOP signs at U-turn crossovers	Left from major street	Left turn at main junction	None	None	None	No
	Major-street through (top of T)	None	None	None	None	No
	Major-street through (stem side)	Through at main junction	None	None	None	No
	Right from major street	Right turn at main junction	None	None	None	No
	Left from minor street	Right turn at main junction	U-turn at crossover stop	None	None	Yes
	Right from minor street	Right turn at main junction	None	None	None	No

Note: EDTT = extra distance travel time to and from U-turn crossover.

For movements with signals at which non-random arrivals are expected, the best way to account for progression is to assemble a complete set of signal data, including offsets, and apply the flow profile procedure from Chapter 18.

For movements with signals at which non-random arrivals are expected, the best way to account for progression is to assemble a complete set of signal data, including offsets, and apply the flow profile procedure from Chapter 18. If detailed signal data are unavailable, analysts should collect arrival type data in the field. Arrival types and field collection of arrival types are discussed in Chapter 18. If neither detailed signal data nor field arrival type data are available, analysts can estimate the arrival type of the second or third signal encountered by using Exhibit 23-51. Default values in Exhibit 23-51 assume that signals at RCUTs and MUTs are timed for optimum progression in both major-street directions and that signals on each side of the RCUT major street are timed independently.

Exhibit 23-51
Default Arrival Types for RCUT and MUT Movements Encountering More Than One Signal

Intersection Type	Movement	Default Arrival Type 2nd Signal	3rd Signal	4th Signal
Four-legged RCUT	Left from major street	3	None	None
	Major-street through	5	None	None
	Right from major street	5	None	None
	Left from minor street	3	5	None
	Minor-street through	3	5	None
Three-legged RCUT	Left from major street	3	None	None
	Left from minor street	3	None	None
Four-legged MUT with signals at U-turn crossovers	Left from major street	5	2	5
	Major-street through	5	None	None
	Right from major street	5	None	None
	Left from minor street	3	5	None
Four-legged MUT with stop signs at U-turn crossovers	Left from major street	3	None	None
	Left from minor street	3	None	None

Note: Use only if it is not possible to apply the Chapter 18 flow profile procedure and no field data are available.

Step 5: Additional Control-Based Adjustments

This step estimates additional control-based adjustments needed to approximate control delay in Step 6. This step is applied for each RCUT or MUT junction.

RCUT Junctions with Merges

The procedure assumes that there is no control delay associated with merging onto the major street or, for vehicles that need to weave, weaving from one side of the major street to the other. Analysts should consider whether this assumption holds for any particular RCUT with merges, since these maneuvers could add travel time.

RCUT and MUT Junctions with STOP Signs

The procedure from Chapter 20, Two-Way STOP-Controlled Intersections, is applied to estimate control delay at junctions with STOP signs. The two additional parameters that must be applied at an RCUT or MUT U-turn crossover with a STOP sign are (*a*) the base critical headway and (*b*) the base follow-up time. Because of the U-turn crossover geometry, it is reasonable to believe that the critical headway and follow-up time from that crossover are different from the left-turn cases presented in Chapter 20. If field data or representative local data for critical headway and follow-up time are unavailable for the U-turn crossover, default values can be applied on the basis of a site with three through lanes and a 55-mi/h speed limit (*12*), where a critical headway of 4.4 s and a follow-up time of 2.6 s were observed. It is reasonable that these values are less than the default values for these parameters given in Chapter 20, since the maneuver is relatively simple, with only one conflicting traffic stream for motorists to observe.

RCUT and MUT Junctions with Traffic Signals

The signalized intersections procedure in Chapter 19 is applied at junctions with signals to estimate saturation flow rates, capacity, and *v*/*c* ratio. Two modifications are made to this procedure to (*a*) adjust the saturation flow rate at U-turn crossovers and (*b*) estimate the effects of right turns and U-turns on red.

Chapter 19 does not provide a U-turn saturation flow rate adjustment factor. Exhibit 23-52 provides default values for this factor for three categories of median width: less than 35 ft, 35 to 80 ft, and greater than 80 ft. The saturation flow rate is lower with narrow medians and higher with very wide medians, because with narrower medians, drivers have to slow to make a sharper U-turn; with wider medians, drivers can maintain higher speeds during a more sweeping U-turn.

Exhibit 23-52
MUT and RCUT Default Saturation Adjustment Factors for U-Turn Crossovers

Median Width	Saturation Flow Rate Adjustment Factor
Narrow (<35 ft)	0.80
Typical (35–80 ft)	0.85
Very wide (>80 ft)	0.95

Source: Hummer (*13*).

Step 6: Estimate Junction-Specific Performance Measures

This step estimates the control delay, *v/c* ratio, and queue lengths at each junction in the RCUT or MUT.

Control Delay and v/c Ratio

The method used to estimate control delay and *v/c* ratio depends on the type of traffic control used for the movement:

- **Merges:** Control delay is assumed to be zero for RCUT junctions with merges, unless Step 5 showed otherwise. Control delay and *v/c* ratio for major-street left turns with YIELD signs must be computed with the Chapter 20 methodology.
- **Stop control:** The Chapter 20 methodology is applied to find control delay and *v/c* ratio.
- **Traffic signals:** The incremental queue accumulation procedure from Chapter 19 is applied to find control delay and *v/c* ratio.

If desired, control delay and *v/c* ratio results for each junction with a STOP sign or signal can be converted into LOS by using the appropriate exhibit from Chapter 20 or Chapter 19, respectively.

Queue Lengths

Queue lengths must be checked by using the procedure from Chapter 19 (for a signal) or Chapter 20 (for a STOP sign) to ensure that queues do not spill back into adjacent through lanes or to another junction:

- **MUTs:** Queue lengths should be checked for the U-turn crossover and for the major street from the main junction back toward the U-turn crossover.
- **RCUTs:** Queue lengths should be checked for the U-turn crossover, for the major street from the main junction back toward the U-turn crossover, and for the left-turn crossover.

If the 95th percentile queue length at any of the above locations exceeds the available storage space, queue spillback is likely to be an issue for a significant portion of the time, and the travel times produced by this method will likely be significantly underestimated.

Step 7: Calculate Extra Distance Travel Time

Step 7 is conducted for each O-D movement that experiences EDTT. Exhibit 23-48 through Exhibit 23-50 showed which movements experience EDTT, for each type of RCUT and MUT addressed by this methodology. For both RCUTs and MUTs, EDTT is always experienced as a "round-trip" from the main junction to the U-turn crossover and back.

RCUTs with Merges

For RCUTs with merges, EDTT is estimated as follows:

$$EDTT = \frac{D_t + D_f}{1.47 \times S_f} + a$$

Equation 23-56

where

$EDTT$ = extra distance travel time (s),

D_t = distance from the main junction to the U-turn crossover (ft),

D_f = distance from the U-turn crossover to the main junction (ft),

1.47 = conversion factor from mi/h to ft/s,

S_f = major-street free-flow speed (mi/h), and

a = delay associated with deceleration into a turn and acceleration from the turn (s).

For minor-street left-turn movements, a is assumed to be 10 s; for a minor-street through movement, it is assumed to be 15 s.

RCUTs and MUTs with STOP Signs and Signals

For RCUTs and MUTs with STOP signs or signals, EDTT is given by

$$EDTT = \frac{D_t + D_f}{1.47 \times S_f}$$

Equation 23-57

where the variables are as defined above. There is no term for acceleration and deceleration in this case, because it is already accounted for in the formula for control delay at junctions with STOP signs or signals. When multiple signals exist along the major street, analysts may measure free-flow speed in the field or use the procedure given in Chapter 18, Urban Street Segments.

Step 8: Estimate Additional Weaving Delay

Step 8 is only used for RCUTs with merges and only when, in the analyst's judgment, there will likely be significant weaving delay. In this case, the analyst must develop an estimate of weaving delay from field measurements or an alternative tool and add it to the EDTT estimate calculated in Step 7.

Step 9: Calculate Experienced Travel Time

ETT is calculated for each O-D movement and is the sum of the control delays experienced, any geometric delay, and EDTT:

$$ETT = \sum d_i + \sum EDTT$$

Equation 23-58

where d_i is the control delay at each junction i encountered on the path through the facility and *EDTT* is the extra distance travel time experienced, with all values in seconds.

Exhibit 23-48 through Exhibit 23-50 showed the junctions traversed by each O-D movement for the various RCUT and MUT intersection types analyzed by this method, and they can be applied when the control delays experienced are compiled. EDTT is obtained from the results of Step 7 and (if applicable) Step 8.

Step 10: Calculate Level of Service

In this step, the ETT for each movement is converted into a LOS by using the criteria given in Exhibit 23-13. If the v/c ratio for any lane group at any junction is greater than 1.0 or if the queue-to-storage ratio exceeds 1.0, the LOS is automatically F, regardless of the ETT value.

Computation of an average ETT for the intersection approach is often desirable. This aggregated ETT is a weighted average ETT, with each movement ETT being weighted by its demand flow rate. This helps establish a context for an RCUT or MUT, where movements that are rerouted to U-turn crossovers often have relatively poor levels of service and are balanced by other movements with relatively good levels of service. The approach ETT is computed with Equation 23-59, with the summations being for all movements on an approach.

Equation 23-59

$$ETT_A = \frac{\sum(ETT_j \times v_j)}{\sum v_j}$$

where

ETT_A = approach experienced travel time (s/veh),

ETT_j = experienced travel time for movement j (s/veh),

v_j = demand flow rate for movement j (veh/h), and

$j \in$ set of all movements on the approach of interest.

Similarly, an intersection ETT can be computed with Equation 23-60, with the summations being for all movements at the intersection.

Equation 23-60

$$ETT_I = \frac{\sum(ETT_k \times v_k)}{\sum v_k}$$

where

ETT_I = intersection experienced travel time (s/veh),

ETT_k = experienced travel time for movement k (s/veh),

v_k = demand flow rate for movement k (veh/h), and

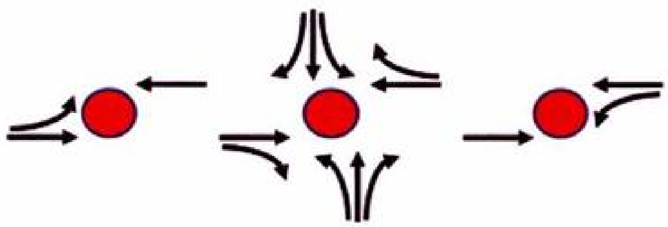
$k \in$ set of all movements at the intersection.

Partial Displaced Left-Turn Intersections

Similar to RCUT and MUT intersections, DLT intersections can be analyzed with the 10-step analysis framework presented in Exhibit 23-47. DLT intersection operations are similar to conventional urban street and signalized intersection operations, and therefore the DLT procedure can be viewed as an extension of the urban streets and signalized intersection procedures. Exhibit 23-53 illustrates the urban street layout of a partial DLT having three signalized intersections. The middle intersection is called the *main intersection.* The other two intersections, where left turns cross over the major street, are the *supplemental intersections.* Major-street left turns are hidden at the main intersection; they are shielded by through movements and do not need to be modeled. For advanced cases where major-street left turns experience delay at the main intersection, alternative tool analysis is recommended.

DLT intersection operations are similar to conventional urban street and signalized intersection operations, and therefore the DLT procedure can be viewed as an extension of the urban streets and signalized intersection procedures.

Exhibit 23-53
Urban Street Layout for a Partial DLT Intersection



Step 1: Determine O-D Demands and Movement Demands

In this step, standard O-D demands are converted into turning movement demands at each component junction. For the most part, this step is performed the same way for DLTs as previously described for RCUTs and MUTs. However, one extra adjustment is needed for displaced left-turn approaches at the main intersection. This extra adjustment is only applicable when the approach's through and left-turning movements are served by the exact same signal phasing and timing. If this condition is met, zero left-turning vehicles are assumed for the approach.

This adjustment is made because displaced left-turn vehicles are not expected to be delayed at the main intersection when the signals are timed properly. This adjustment is necessary because the Chapter 19 signalized intersection procedure, applied in Step 6, cannot simultaneously move protected-phase left turns and opposing through vehicles. In cases where displaced left-turn vehicles are significantly delayed at the main intersection or in cases where the approach's through and left-turning movements are not served by the exact same signal phasing and timing, use of an alternative tool (most likely a microsimulation tool) is recommended.

Steps 2 and 3: Determine Lane Groups and Lane Utilization

Steps 2 and 3 are performed in accordance with the lane group determination and lane utilization procedures described in Chapter 19, Signalized Intersections.

Step 4: Signal Progression Adjustments

In Step 4, rather than adjusting the arrival types, a complete flow profile analysis should be performed as described in Chapter 18, Urban Street Segments. The flow profile analysis will estimate the proportion of vehicles arriving on green.

Step 5: Additional Control-Based Adjustments

The adjustments made during Step 5 provide for better estimation of control delay in Step 6. At the supplemental intersections, the Chapter 19 right-turn saturation flow rate adjustment factor (typically $f_{RT} = 0.85$) should be applied to left-turning vehicles crossing over the opposing through lanes.

Two validity checks should be made during this step. The first check is that green times at the main intersection should always be large enough to serve displaced left-turning vehicle demands fully. The second check applies to a low-volume side street scenario or any scenario with relatively short green times on the side street, where the main street green may be started before left turners arrive and may be terminated before left-turning platoons are served. Because the DLT method assumes that the entire left-turning platoon is fully served on every cycle, side street green durations should exceed the sum of (*a*) main street travel time between the supplemental and main intersections and (*b*) displaced left-turn queue clearance time. If these two checks are not satisfied, this method is less reliable, and alternative tool analysis might be needed.

An important signal-timing adjustment is also made during this step. The adjustment is only applicable for displaced left-turn approaches at the main intersection and only when through and left-turning movements are served by the exact same signal phasing and timing on that approach. If these conditions are met, offsets should be set so that displaced left-turn vehicles always arrive during the guaranteed green window at the main intersection. The offset computation is based on several factors, including free-flow travel times, phase durations at each intersection, reference phases at each intersection, reference points at each intersection, and the background system cycle length. The basic computation process for offsets is provided below in Equation 23-61 through Equation 23-66; Example Problems 5 and 6 in Chapter 34 illustrate the offset computation process.

1. Determine the travel distance from upstream stop line to downstream stop line for the displaced left-turn roadway TD_{DLT}, in feet. The displaced left-turn roadway is the roadway used by displaced left-turning vehicles as they travel from the upstream crossover at the supplemental intersection to the stop bar at the main intersection.
2. Compute the left-turn travel time TT_{DLT}, in seconds, from the free-flow speed of the displaced left-turn roadway $S_{f,DLT}$, in miles per hour; this calculation includes a conversion from miles per hour to feet per second:

Equation 23-61

$$TT_{DLT} = \frac{TD_{DLT}}{S_{f,DLT} \times 1.47}$$

3. For the upstream supplemental intersection, obtain the duration between the reference point and the start of the displaced left-turn phase LAG_{DLT}, in seconds. For the downstream main intersection, obtain the duration between the reference point and the start of the major-street through phase LAG_{TH}, in seconds. These durations should be based on input phase splits instead of output phase durations.
4. Obtain the offsets at the upstream supplemental intersection O_{SUPP} and the downstream main intersection O_{MAIN}, both in seconds.
5. Compute the system start time of the displaced left-turn phase ST_{DLT}, in seconds, for the upstream crossover at the supplemental intersection:

$$ST_{DLT} = LAG_{DLT} + O_{SUPP}$$

Equation 23-62

6. Compute the system start time of the major-street through phase ST_{TH} at the main intersection, in seconds:

$$ST_{TH} = LAG_{TH} + O_{MAIN}$$

Equation 23-63

7. Change O_{SUPP} so that ST_{TH} is equal to $ST_{DLT} + TT_{DLT}$:

$$O_{SUPP}(\mathrm{s}) = O_{SUPP} - ST_{DLT} + ST_{TH} - TT_{DLT}$$

Equation 23-64

8. If the offset value is greater than the background cycle length value, decrement the offset value by the cycle length C to obtain an equivalent offset within the valid range:

$$\text{If } O_{SUPP} > C \text{ then } O_{SUPP} = O_{SUPP} - C$$

Equation 23-65

9. If any offset value is lower than zero, increment the offset value by the cycle length to obtain an equivalent offset within the valid range:

$$\text{If } O_{SUPP} < 0 \text{ then } O_{SUPP} = O_{SUPP} + C$$

Equation 23-66

Step 6: Estimate Junction-Specific Performance Measures

Offsets computed in Step 5 will influence the proportion of vehicles arriving on green (PVG), according to Chapter 18's methods. This PVG will then affect the computation of control delay and *v/c* ratio at each component intersection, according to Chapter 19's methods.

Steps 7 and 8: Calculate Extra Distance Travel Time and Additional Weaving Delay

EDTT and weaving delay are assumed to be negligible for DLT intersections.

Step 9: Calculate Experienced Travel Time

ETT is calculated for each O-D movement in the same way as described previously for RCUTs and MUTs.

Step 10: Calculate Level of Service

LOS for the DLT intersection is based on ETT. A weighted average ETT is computed for the intersection by using Equation 23-67. To avoid double-counting vehicle trips inside the spatial boundaries (see Exhibit 23-5), the summation of O-D demand volumes v_{OD} should be identical to what it would be for a conventional intersection.

Equation 23-67

$$ETT_{DLT} = \frac{\sum(d_j \times v_j)}{\sum v_{OD}}$$

where

ETT_{DLT} = weighted average experienced travel time for the DLT intersection (s/veh),

d_j = control delay for movement j (s/veh),

v_j = demand volume for movement j (veh/h), and

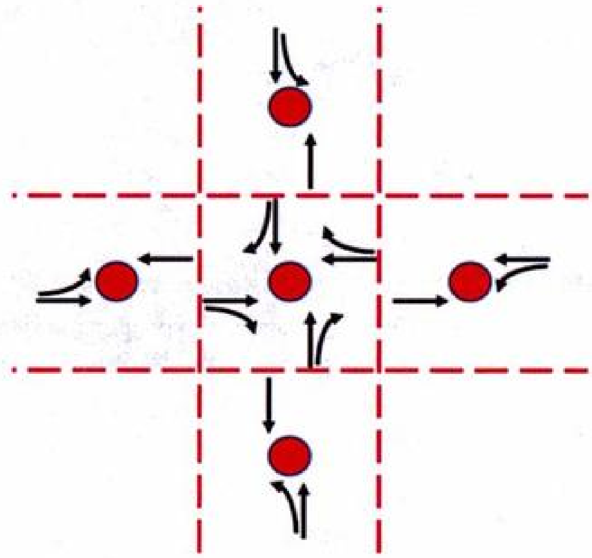
v_{OD} = O-D demand volumes (veh/h).

Full DLT Intersections

Overview

The analysis of a full DLT intersection involves aggregating the results of two partial DLT analyses (one in the east–west direction and one in the north–south direction). Exhibit 23-54 illustrates both partial DLT analyses, which share one main intersection. Channelized right-turn lanes (separated from the main intersection) and the demand volumes that use channelized right-turn lanes should be omitted from the analysis.

Exhibit 23-54
Urban Street Layout for a Full DLT Intersection



Pseudo–Right-Turn Assignment

The "pseudo–right-turn" concept exists mainly to overcome fundamental limitations in the HCM computational framework, in which protected left turns cannot move together with opposing through movements. An extra step for the analysis of full DLT intersections is to assign pseudo–right-turn movements to one or more approaches to the main intersection. This extra step compensates for the prior assumption of zero displaced left-turn flows at the main intersection, which was necessary for accurate modeling of the main intersection. Unless the pseudo–right-turn technique is used, this assumption will lead to incorrect modeling of downstream intersections. Pseudo–right turns allow for proper flow balancing and flow profiles at downstream intersections (*14*). Delays are not tabulated for pseudo–right turns, because the displaced left-turning vehicles they represent are typically not delayed at the main intersection.

Pseudo–right turns were not needed for partial DLTs, because there was no concern over incorrect modeling of downstream intersections. Within the spatial boundaries of a partial DLT analysis, no downstream intersections exist for displaced left turners once they pass through the main intersection. However, at full DLTs, displaced left turners are sometimes stopped at a downstream supplemental intersection. In the advanced design under which opposing left-turn vehicles merge into the channelized right-turn lane and bypass the downstream supplemental intersection without stopping, pseudo–right turns should be omitted from the analysis.

In the example illustrated in Exhibit 23-55, pseudo–right turns are defined on the southbound approach. Displaced left turns on the northbound approach are then removed from the analysis. Pseudo–right turns should only be defined at the main intersection and not at the supplemental intersections. Traffic demand and number of lanes should match the displaced left turns. For maximum accuracy of the downstream flow profiles, the Chapter 19 saturation flow adjustment factor for left turns (typically $f_{LT} = 0.95$) should be applied to the pseudo–right turns. Start-up lost times should be set to zero for the pseudo–right turns to reflect the uninterrupted flow of displaced left turns.

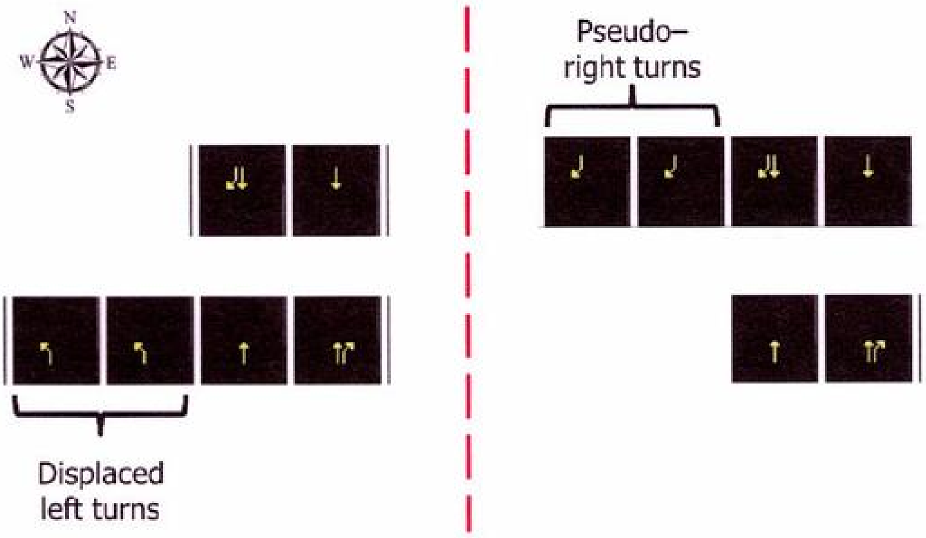


Exhibit 23-55
Example Conversion of Displaced Left Turns to Pseudo–Right Turns

Signal Timing Considerations

When signal timing for the two partial DLT analyses is defined, the same background cycle length should be used at all intersections. Furthermore, the main intersection should have exactly the same signal timing in both partial DLT analyses. Finally, the main intersection should have fixed-time control (no actuation), to allow guaranteed green windows along both major and minor streets. These guaranteed green windows allow displaced left-turning vehicles on any approach to move through the main intersection without stopping. Actuation at the supplemental intersections is feasible and will not reduce guaranteed green windows at the main intersection.

Performance Measure Calculation

To avoid double-counting delays and to account for flow profiles properly, minor-street performance measures should not be tabulated in the partial DLT analyses. Instead, major-street performance measures from both analyses are combined and averaged. Example Problems 16 and 17 in Chapter 34 illustrate the volume-weighted averaging of control delays.

4. EXTENSIONS TO THE METHODOLOGY

EVALUATION OF OTHER ALTERNATIVE INTERSECTION FORMS

The methodology described in Section 3 can be extended to other types of RCUT and MUT intersections, including the following:

- An RCUT with direct minor-street left turns and rerouted major-street left turns (*15*),
- An RCUT with no direct left turn,
- An MUT with three or four U-turn crossovers, and
- An MUT with one or two direct left turns allowed ("partial MUT").

In all of these cases, the basic analysis framework remains the same. The control delay can be estimated at each junction for each movement, the EDTT can be estimated for each link for rerouted vehicles, and the ETT can be estimated as the sum of those components.

Some RCUTs and MUTs provide a two-way driveway or side street at the end of the U-turn crossover, as illustrated in Exhibit 23-56. The side street is in the upper-left corner of Exhibit 23-56. When demand from the side street is substantial, the traffic stream from the crossover behaves like a permissive left turn from a shared left and through lane, and the corresponding methodology from Chapter 19 should be applied to estimate the saturation flow rate of the crossover lane group.

Exhibit 23-56
Side Street at the End of an MUT U-Turn Crossover in Michigan

Source: © 2015 Google.

The concepts presented in this chapter could also be used to analyze other types of alternative intersections that have been built in the United States. For example, *jughandle intersections* use right-side ramps to reroute left turns from the major street. The *quadrant roadway intersection* reroutes all four left turns at a four-legged intersection to a connector between the two intersecting roadways (*7*). Finally, the *continuous green T-intersection* is a three-legged design in which the major-street through movement on top of the T is separated from the rest of the intersection and does not travel through the traffic signal (*7*).

PEDESTRIAN AND BICYCLE ANALYSIS

Minor-Street Crossings at Signalized RCUTs

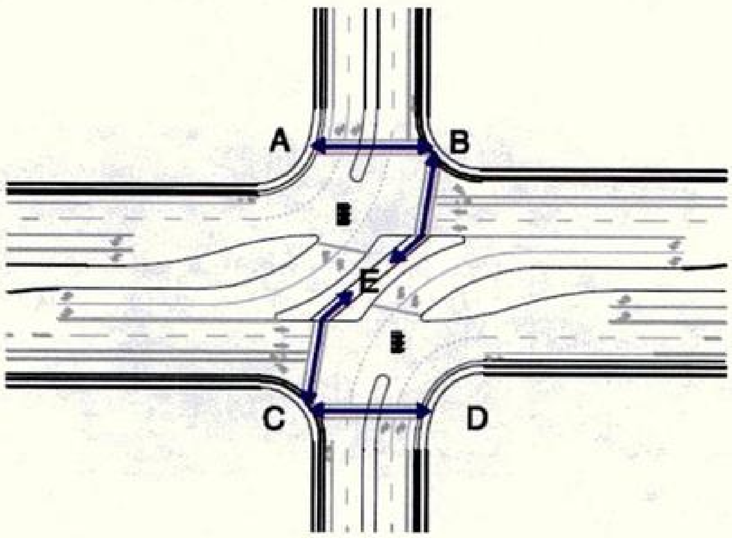
At an RCUT with signals, the experience for pedestrians and bicyclists crossing the minor street is, from an operational standpoint, not significantly different from their experience at a conventional intersection. Therefore, the operational analysis procedures for pedestrians and bicyclists from Chapter 19, Signalized Intersections, can be applied directly for those movements.

Major-Street Crossings at Signalized RCUTs

Exhibit 23-57 shows the typical pedestrian crossing designs at a signalized four-legged RCUT. For example, a northbound pedestrian crossing the major street on the east side of the minor street must make a three-stage crossing, from D to C, then from C to E, and finally from E to B. The movement from C to B will almost always be in two stages, with the pedestrian likely waiting for the WALK signal from E to B, because the signals on each side of the RCUT main street are typically timed independently of each other.

During each of the three crossing stages, the operational analysis procedures for pedestrians and bicyclists from Chapter 19 can be applied directly. The complete three-stage movement will then receive three individual sets of performance measures and LOS.

Exhibit 23-57
Typical Pedestrian Crossing of a Four-Legged Signalized RCUT



Source: Hummer et al. (*6*).

Exhibit 23-58 shows the typical pedestrian crossing designs at a signalized three-legged RCUT. For example, a northbound pedestrian crossing the major street on the east side of the minor street must make a three-stage crossing, from D to C, then from C to E, and finally from E to A; a northbound pedestrian crossing the major street on the west side of the minor street must make a two-stage crossing, from C to E and then from E to A. Exhibit 23-58 shows that the crossing from E to A is signalized. Signals such as this on RCUTs and MUTs are typically easy to coordinate with other signals along the arterial and introduce minimal vehicle delay.

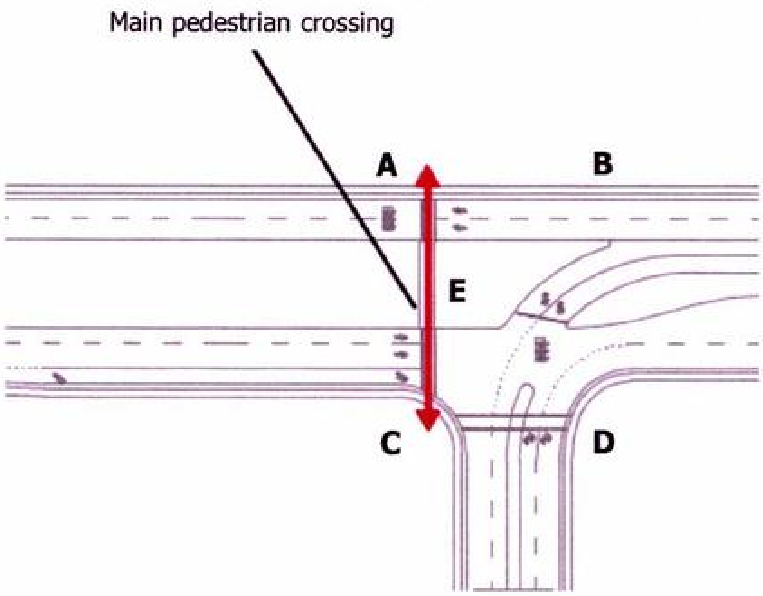


Exhibit 23-58
Typical Pedestrian Crossing of a Three-Legged Signalized RCUT

Source: Hummer et al. (*6*).

At a typical signalized RCUT, a bicyclist crossing the major street has a choice of using the pedestrian crossing as illustrated in Exhibit 23-57 and Exhibit 23-58 (whether riding or dismounted) or of using the vehicle path to and from the U-turn crossover. In either case, the methods described in Chapter 19 to estimate performance measures and LOS for bicycles crossing the intersection do not apply.

Crossings at MUTs

At an MUT, the operational analysis procedures for pedestrians and bicycles from Chapter 19 can be applied directly. The pedestrian LOS score equation (Equation 19-71) and the bicycle LOS score equation (Equation 19-79) were not developed from user experiences at MUTs, but the pedestrian and bicycle crossing experiences at MUTs are not that different from conventional intersections, so the results should still be useful.

Crossings at DLTs

DLT pedestrian crossings differ significantly from the pedestrian crossings at conventional intersections, and these differences can affect both pedestrian and vehicular delay (*8*). As a result, the Chapter 19 analysis procedures for pedestrians and bicycles are not applicable to DLT intersections. Instead, there are two basic methods for handling pedestrians at a DLT. One method prioritizes pedestrian safety over displaced left-turning vehicle operations. The second method does the opposite, by favoring vehicles (*8*).

In the first method, pedestrians can pass between the four outer corners of the intersection, similar to a conventional intersection. However, this method would often require displaced left-turning vehicles to stop at the main intersection, which would defeat the purpose of continuous flow. The only way to avoid stopping displaced left-turn vehicles would be for pedestrians to receive the WALK signal after left-turning platoons had cleared. This might not be possible under all combinations of vehicular and pedestrian demand. In addition, the combination of long pedestrian crossings (e.g., requiring 40 s of walk time) and the need for separate pedestrian phases can sometimes (*a*) require inefficient longer cycle lengths and (*b*) cause pedestrians to be the critical movement affecting signal timing design.

In the second method, pedestrians must move between safe "refuges" on their way across the intersection. This prevents any conflict between pedestrians and displaced left-turning vehicles. In some cases, pedestrians would endure high-speed vehicle movements on both sides of the refuge area. The second method is illustrated in Exhibit 23-59.

Exhibit 23-59
Two-Stage Pedestrian Crossing at a DLT

Source: Steyn et al. (*8*).

5. APPLICATIONS

EXAMPLE PROBLEMS

Exhibit 23-60 lists the example problems presented in Chapter 34, Interchange Ramp Terminals: Supplemental.

Exhibit 23-60
Listing of Alternative Intersection Example Problems Contained in Chapter 34

Example Problem	Description	Application
12	Four-legged RCUT with merges	Operational
13	Three-legged RCUT with STOP signs	Operational
14	Four-legged RCUT with signals	Operational
15	Four-legged MUT with STOP signs at U-turn crossovers	Operational
16	Partial DLT with signals (displaced left turns on two approaches)	Operational
17	Full DLT with signals (displaced left turns on four approaches)	Operational

EXAMPLE RESULTS

This section presents the results of applying this chapter's method in typical situations.

DLT Intersections

The relevant input parameters and the sensitivity of results to those inputs are much the same for DLTs as for conventional intersections. However, comparison of facility type performance at various traffic demand levels can be informative. Exhibit 23-61 compares average control delay per vehicle for a conventional intersection with that of an intersection having displaced left turns on one or more approaches. Original turn movement volumes are represented by a demand multiplier of 100%. These demands were then multiplied by 25%, 50%, 75%, and 125%. Intersection conditions used to generate Exhibit 23-61 involved heavy left-turn demands on the major street and low demands on the side street. Lane geometry for the DLT was designed to match lane geometry for the conventional intersection, to the extent possible. Turn movement demands were also identical. Results suggest that the partial DLT configuration would be efficient under this specific set of volume demands.

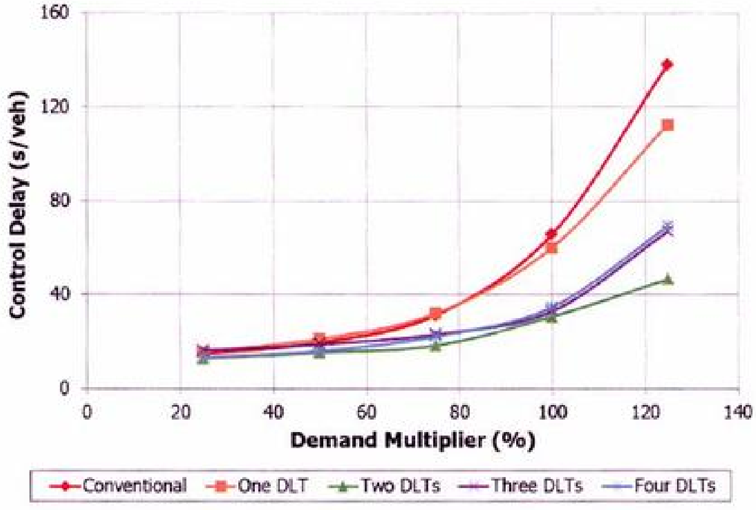


Exhibit 23-61
Example Results Comparing DLT and Conventional Intersection Performance

Exhibit 23-64
Sensitivity of LOS to Changes in Minor-Street Demand

Demand (veh/h)		Control Delay (s/veh)		LOS	
Minor-Street Left Turn	Minor-Street Right Turn	Minor-Street Approach	U-Turn Crossover	Minor-Street Left Turn	Minor-Street Right Turn
25	25	11.6	9.2	D	B
50	50	12.3	9.4	D	B
100	100	14.8	9.7	D	B
150	160	19.4	10.0	D	B
200	200	27.6	10.5	D	C
250	250	51.0	11.0	E	D
300	300	104.3	11.6	F	F

Right Turns and U-Turns on Red

The performance of MUT and RCUT intersections is particularly aided by right turns and U-turns on red, because demands for those movements are higher than at conventional intersections. Exhibit 23-65, based on Example Problem 15 in Chapter 34, shows the effect on ETT at a MUT when right turns on red are allowed from the minor-street approaches (the minor street contains the eastbound and westbound approaches) with exclusive right-turn lanes. If 40% of the right-turning volume (which includes the traffic that will eventually turn left) is able to turn on red with an estimated zero control delay, ETT will be reduced by more than 11 s/veh for some of the minor-street movements. This will affect LOS in some cases.

Exhibit 23-65
Sensitivity of ETT to Percentage of Minor-Street Traffic Turning Right on Red

Movement	ETT (s/veh)				
	0% RTOR	10% RTOR	20% RTOR	30% RTOR	40% RTOR
NB left	78.0	77.9	77.7	77.6	77.4
SB left	56.1	55.9	55.8	55.6	55.4
NB through	9.3	9.3	9.2	9.2	9.1
SB through	12.3	12.2	12.1	12.0	11.9
NB right	9.4	9.3	9.3	9.2	9.1
SB right	13.7	13.6	13.5	13.4	13.3
EB left	67.4	64.3	61.4	58.7	56.1
WB left	87.5	85.2	82.9	80.7	78.5
EB through	25.1	25.2	25.3	25.3	25.4
WB through	22.2	22.2	22.3	22.3	22.4
EB right	23.7	20.6	17.8	15.1	12.6
WB right	20.2	18.0	15.8	13.7	11.6

Note: NB = northbound, SB = southbound, EB = eastbound, WB = westbound, RTOR = right turn on red.

PLANNING-LEVEL ANALYSIS

Intersections with Signal Control

The methodology described in Section 3 is for the operational analysis of an RCUT, MUT, or DLT. A planning-level analysis for any of the three designs with signal control is possible and may be useful when detailed data are unavailable. In this case, analysts should use the simplified method for determining the critical intersection volume-to-capacity ratio X_c, described in Chapter 32, Signalized Intersections: Supplemental, at each signalized junction associated with the intersection. The junction with the highest volume-to-capacity ratio will be "critical" to overall intersection performance. The highest v/c ratio at any point at an intersection is a useful predictor of overall intersection performance at a planning level, although some factors affecting operational performance, such as some nuances of signal control and operational effects of geometrics, are not captured (*6*). The simplified method for determining the v/c ratio at each junction also will not capture the EDTT experienced by rerouted vehicles.

Intersections with STOP Control

A planning-level analysis of RCUTs and MUTs with STOP-controlled junctions can be performed by applying the planning-level procedure described in Chapter 20, Two-Way STOP-Controlled Intersections, at each of the STOP-controlled junctions. The Chapter 20 planning method uses all of the geometric and traffic 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 analyses are performed.

RCUT Intersections with Merges

A planning analysis of a merge-controlled RCUT intersection applies the Chapter 20 planning-level procedure for STOP-controlled intersections at the two YIELD-controlled crossovers to determine whether either is at or near capacity. Such an approach is conservative, since a YIELD-controlled movement would have a higher capacity than a STOP-controlled movement. However, even at an early project development stage, most analysts are likely to conduct an operational analysis rather than a planning analysis, because the effort to do so is only incrementally greater (*6*).

6. REFERENCES

Some of these references can be found in the Technical Reference Library in Volume 4.

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CHAPTER 24
OFF-STREET PEDESTRIAN AND BICYCLE FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

Off-street pedestrian and bicycle facilities (*a*) are used only by nonmotorized modes and (*b*) are not considered part of an urban street or transit facility. Thus, a shared-use path only 10 ft from a roadway but separated by a sound barrier may be considered an off-street facility, but a sidepath separated from the roadway by a 10-ft planted buffer would generally be considered an on-street facility. Facilities located directly along an urban street (e.g., bicycle lanes, sidewalks) are not addressed in this chapter. In general, 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 on off-street facilities.

Facilities located within approximately 35 ft of an urban street are generally 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 18, Urban Street Segments. The definition also excludes crosswalks and queuing areas; these areas are addressed in the intersection chapters (Chapters 19–23). 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.

Off-street facilities are generally located more than 35 ft from a roadway, although the exact distance may vary on the basis of the local context.

Pedestrian and bicycle facilities along urban streets are addressed in Chapters 16 and 18.

Bicycle facilities along multilane and two-lane highways are addressed in Chapter 15.

The Transit Capacity and Quality of Service Manual *covers the analysis of pedestrian facilities serving transit stops and stations.*

CHAPTER ORGANIZATION

Chapter 24 provides capacity and level-of-service (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.

Descriptions and illustrations of each of the above facility types are provided in Chapter 3, Modal Characteristics.

Chapter 24 is divided into five sections. Section 2 defines LOS criteria for off-street pedestrian and bicycle facilities. Section 3 provides the core methodologies for evaluating the operation and quality of service of off-street pedestrian walkways and stairways, bicycle paths, and shared-use paths. Section 4 provides guidance on extending these methods to pedestrian streets and plazas, walkways with grades exceeding 5%, and off-street facilities segregating pedestrians and bicyclists. Section 5 presents guidance on using the results of an off-street facility analysis, including example results from the methods, information on the sensitivity of results to various inputs, and potential applications.

RELATED HCM CONTENT

Other HCM content related to this chapter includes

- Chapter 3, Modal Characteristics, which introduces the pedestrian and bicycle modes, provides examples of off-street pedestrian and bicycle facility types, and discusses the variability of bicycle demand;
- Chapter 4, Traffic Operations and Capacity Concepts, which includes sections on pedestrian and bicycle flow and capacity;
- Chapter 35, Pedestrians and Bicycles: Supplemental, which provides example problems with step-by-step calculations applying the off-street pedestrian and bicycle facility methods; and
- Section O, Pedestrians, Bicycles, and Public Transit, of the *Planning and Preliminary Engineering Applications Guide to the HCM*, found in online Volume 4, which provides guidance on incorporating this chapter's methods and performance measures into a planning effort.

COMPUTATIONAL ENGINE

The Federal Highway Administration research that developed the shared-use path method (5) presented in this chapter also developed a spreadsheet-based computational engine for the method. A modified version of the original engine is available in the Chapter 24 section of the Technical Reference Library in online Volume 4. The original research applied the peak hour factor (PHF) at a different point in the calculation process than it is applied in the HCM methods. The version of the engine posted in Volume 4 applies the PHF as described in this chapter. Users should note that the engine's fixed segment length of 3 mi cannot be changed by the user without significant modifications to the engine.

2. CONCEPTS

LOS CONCEPTS

Pedestrian and bicycle quality-of-service measures for off-street facilities differ from those for on-street facilities. On-street quality of service, as described in the previous chapters in Volume 3, strongly reflects the effects of motorized traffic on nonmotorized travelers' perceptions of comfort and safety. However, by definition, motorized traffic effects are absent on off-street facilities. Instead, quality of service for off-street facilities reflects the interactions of facility users with each other.

LOS CRITERIA

Three service measures are defined in this chapter. The measure(s) to apply to an analysis depend on the travel mode and type of off-street facility:

- For pedestrians on exclusive pedestrian facilities, the service measure is *pedestrian space*, measured in square feet per pedestrian;
- For pedestrians on facilities shared by pedestrians and bicycles, the service measure is the number of *bicycle meeting and passing events per hour*; and
- For bicycles on both shared-use and exclusive paths, the service measure is a *bicycle LOS* (BLOS) *score* incorporating meetings per minute, active passings per minute, presence of a centerline, path width, and delayed passings.

In the case of the pedestrian space measure, different LOS thresholds apply, depending on the type of facility under study and, in some situations, the nature of the pedestrian flow along the facility. LOS thresholds for pedestrian facilities in a transit station context, as given in the *Transit Capacity and Quality of Service Manual* (1), allow for higher levels of crowding for a given LOS than do the thresholds for off-street pedestrian facilities.

LOS thresholds for off-street pedestrian and bicycle facilities are based on available user perception research 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. For the purposes of evaluating LOS, a "pedestrian" is considered to be someone who is walking; therefore, pedestrian LOS does not necessarily reflect the quality of service experienced by joggers, persons in wheelchairs, or others who could also be considered pedestrians.

LOS does not reflect whether a facility complies with the ADA or other standards.

Walkways

The walkway LOS tables apply to paved pedestrian paths, pedestrian zones (e.g., exclusive pedestrian streets), walkways and ramps with up to a 5% grade, and pedestrian walking zones through plaza areas. Exhibit 24-1 applies when pedestrian flow along the facility is random. Exhibit 24-2 applies when platoons of pedestrians form along the facility, for example, when a signalized crosswalk is located at one end of the segment being analyzed, or when the walkway's operation during special events is 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 square feet per person (ft^2/p), as indicated in the notes for Exhibit 24-1 and Exhibit 24-2.

Exhibit 24-1
Random-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 24-1 does not apply to walkways with steep grades (>5%). See Section 4 for further discussion. v/c = volume to capacity.
[a] Pedestrians per minute per foot of walkway width.
[b] v/c ratio = flow rate/23, based on random flow. LOS is based on average space per pedestrian.
[c] In cross-flow situations, the LOS E–F threshold is 13 ft^2/p.

Exhibit 24-2
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 24-3 provides the LOS criteria for stairways.

Exhibit 24-3
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 24-4 shows pedestrian LOS criteria for paths shared between pedestrians and bicycles.

Exhibit 24-4
Pedestrian LOS Criteria for Shared-Use Paths

LOS	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 flow rates (i.e., flow during the peak 15 min) 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 24-5 provides LOS criteria for bicycles on both shared-use and exclusive off-street paths.

Exhibit 24-5
LOS Criteria for Bicycles on Shared-Use and Exclusive Paths

LOS	BLOS 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

3. CORE METHODOLOGIES

SCOPE OF THE METHODOLOGIES

Pedestrian and bicycle facilities along urban streets are addressed in Chapters 16 and 18.

Bicycle facilities along multilane and two-lane highways are addressed in Chapter 15.

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.

Procedures for estimating LOS are divided into three categories: pedestrians on exclusive pedestrian facilities, pedestrians on shared-use paths, and bicyclists on both shared-use paths and exclusive bicycle facilities.

Much of the material in this section 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 (*5*).

Spatial and Temporal Limits

The analysis of off-street pedestrian and bicycle facilities occurs at the segment level.

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 occurs:

- 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., a walkway becomes a stairway).

The analysis period length is 15 min.

In most cases, the minimum segment length will be around 0.25 mi, and the maximum segment length will be 2 to 3 mi (*5*). Certain kinds of facilities, such as stairways, cross-flow areas, and pedestrian plazas, will have shorter segment lengths. The analysis period length is 15 min.

Performance Measures

These methods produce the following performance measures:

- Average pedestrian space on exclusive pedestrian facilities;
- Weighted event rate for pedestrians on shared-use facilities, where an "event" is a bicycle meeting or passing a pedestrian;
- BLOS score for bicyclists on off-street facilities that reflects bicyclists' perceptions of the facility's operational quality;
- LOS derived from the above three measures;
- Pedestrian volume-to-capacity ratio on exclusive pedestrian facilities; and
- Rates of bicyclists meeting, actively passing, and being delayed in passing other off-street facility users.

Limitations of the Methodologies

Each facility type 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 methodologies do not address LOS for pedestrians with disabilities such as vision or mobility impairments. The reader is encouraged to consult material published by the United States Access Board to ensure compliance with the ADA.

These methodologies do not consider the continuity of walkways, bikeways, and shared-use paths when determining 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 for nonmotorized travel and do not assess the impact of intersections with motorized vehicle facilities.

The methodology does not address the impact of roadway crossings on the LOS of off-street paths.

The methodologies are based solely on facility characteristics and do not consider external factors such as weather, landscaping, adjacent land uses, and lighting conditions, which may also affect users' perceptions of a facility.

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 lower than capacity. For this reason, off-street walkways are desirably designed to achieve LOS C or better (i.e., to achieve uncrowded walking conditions within the designated path area), rather than LOS E (i.e., capacity). 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. Off-street pathways are desirably designed to achieve LOS C or better to avoid this situation.

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.

Shared-Use Paths

The methodology for shared-use paths does not account for the effect on pedestrian LOS of path width or the effects 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 pedestrian shared-use path methodology does not account for the effects of nonbicyclist users of the path on pedestrian LOS.

The methodology for BLOS on shared-use paths incorporates the effects of five user groups: bicyclists, pedestrians, runners, inline skaters, and child bicyclists. However, several 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 Section 4.

REQUIRED DATA AND SOURCES

Exhibit 24-6 lists the information necessary to apply the methodologies and suggests potential sources for obtaining these data. It also suggests default values for use when specific information is not available (*5*). The user is cautioned that every use of a default value instead of a field-measured value may make the analysis results more approximate and less related to the specific conditions that describe the facility. HCM defaults should only be used when (*a*) field data cannot be collected, and (*b*) locally derived defaults do not exist.

In particular, service measure results are moderately sensitive to the choice of PHF and highly sensitive to the choice of directional factor. Use of field-measured peak 15-min volumes by direction avoids the need to apply these factors. In addition, the service measure for exclusive pedestrian facilities is highly sensitive to the average pedestrian speed input into the method.

Exhibit 24-6
Required Input Data, Potential Data Sources, and Default Values for Off-Street Facility Analysis

Required Data and Units	Mode and Facility Type: Ped. (Excl.)	Ped. (Shared)	Bike (All)	Potential Data Source(s)	Suggested Default Value
Facility width (ft)	•		•	Field data, aerial photo	Must be provided
Effective facility width (ft)	•			Field data, aerial photo	Same as facility width
Pedestrian volume (p/h)	•			Field data	Must be provided
Bicycle volume (bicycles/h)		•		Field data	Must be provided
Total path volume (p/h)			•	Field data	Must be provided
Bicycle mode split (decimal)			•	Field data	0.55 (i.e., 55% of total path volume)
Pedestrian mode split (decimal)			•	Field data	0.20
Runner mode split (decimal)			•	Field data	0.10
Inline skater mode split (decimal)			•	Field data	0.10
Child bicyclist mode split (decimal)			•	Field data	0.05
Peak hour factor (decimal) [a]	•	•	•	Field data	0.85
Directional volume split (decimal) [b]		•	•	Field data	0.50
Average pedestrian speed (ft/min)	•			Field data	300 ft/min
Average pedestrian speed (mi/h)		•	•	Field data	3.4 mi/h
Pedestrian speed SD (mi/h)			•	Field data	0.6 mi/h
Average bicycle speed (mi/h)		•	•	Field data	12.8 mi/h
Bicycle speed SD (mi/h)			•	Field data	3.4 mi/h
Average runner speed (mi/h)			•	Field data	6.5 mi/h
Runner speed SD (mi/h)			•	Field data	1.2 mi/h
Average inline skater speed (mi/h)			•	Field data	10.1 mi/h
Inline skater speed SD (mi/h)			•	Field data	2.7 mi/h
Average child bicyclist speed (mi/h)			•	Field data	7.9 mi/h
Child bicyclist speed SD (mi/h)			•	Field data	1.9 mi/h
Segment length (mi)			•	Field data, aerial photo	Must be provided
Walkway grade ≤ 5% (yes or no) [c]	•			Field data	Must be provided
Pedestrian flow type (random or platooned) [d]	•			Field data	Must be provided
Centerline stripe (yes or no)			•	Field data	Must be provided

Notes: Ped. = pedestrian; Excl. = exclusive; SD = standard deviation.
Bold italic indicates high sensitivity (>20% change) of service measure to the choice of value.
Bold indicates moderate sensitivity (10% to 20% change) of service measure to the choice of value.
[a] Not required for pedestrian analysis when peak 15-min demand volumes are provided.
[b] Not required when directional demand volumes are provided.
[c] Pedestrian speeds reduce when grades exceed 5%; the service measure is highly sensitive to average pedestrian speed.
[d] LOS letter result is highly sensitive to the selection of pedestrian flow type.

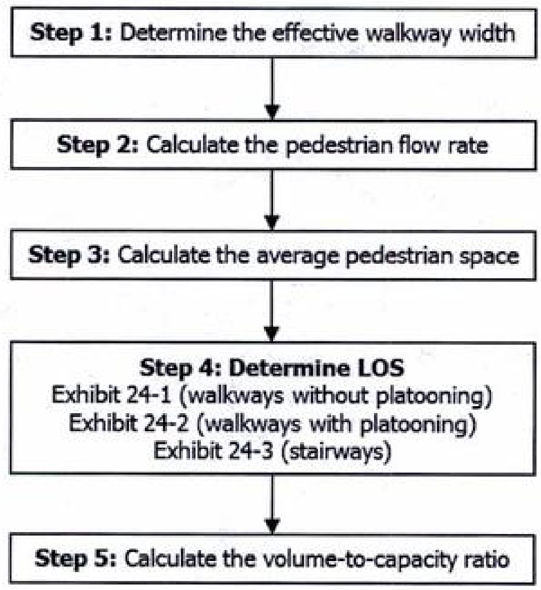
Source: Default values from Hummer et al. (*5*), except for effective facility width.

EXCLUSIVE OFF-STREET PEDESTRIAN FACILITIES

Overview

Exhibit 24-7 illustrates the steps required for the exclusive off-street pedestrian facility methodology.

Exhibit 24-7
Flowchart for Analysis of 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 pedestrians can effectively use. The effective walkway width at a given point along the walkway is computed as follows:

Equation 24-1

$$W_E = W_T - W_O$$

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 pedestrians leave between themselves and linear objects along a walkway, such as curbs and building faces.

Exhibit 24-8 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 an associated *shy distance*, which is the buffer 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.

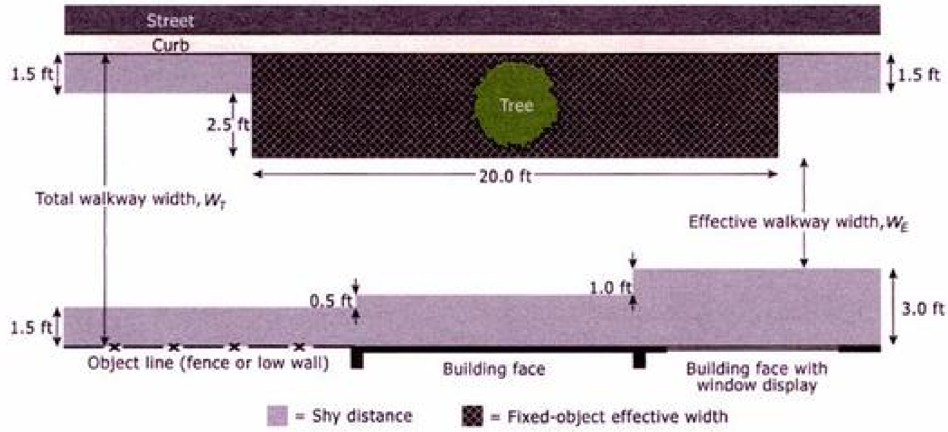


Exhibit 24-8
Width Adjustments for Fixed Objects and Linear Features

The concept of effective width applies to both on-street and off-street facilities. Because of the proximity of the street in Exhibit 24-8, the sidewalk here would be considered an on-street pedestrian facility.

The street is shown so all factors that can influence the effective width of walkways can be depicted in one place.

Exhibit 24-8 also shows that the effective width of a fixed object (here, a tree) 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 because 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 24-9 gives the effective widths of a variety of typical fixed objects found along on- and off-street pedestrian facilities. The values in Exhibit 24-9 can be used when specific walkway configurations are not available.

Stairways

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 people can form on the stair and the side-to-side spacing between them, which affect both the ability of faster pedestrians to pass slower-moving pedestrians and the level of interference between adjacent lines of people. Consequently, meaningful increases in capacity are not linearly proportional to the width but occur in increments of about 30 in. (*1*).

Pedestrians tend to walk in lines or lanes on stairways; thus, meaningful increases in capacity are related to the number of pedestrian lanes available.

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 24-1.

Small reverse flows on stairways should be assumed to use one pedestrian lane (30 in.) of width.

Exhibit 24-9
Typical Fixed-Object Effective Widths

See Exhibit 24-8 for shy distances associated with curbs and building faces.

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 × 1.7 ft)	3.2–3.7
Telephone booths (2.7 ft × 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*).

Step 2: Calculate Pedestrian Flow Rate

Walkways and Cross-Flow Areas

Hourly pedestrian demand 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 24-2

$$v_{15} = \frac{v_h}{4 \times PHF}$$

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.

If peak 15-min pedestrian volumes are available, the highest 15-min volume can be used directly in the method without the application of a PHF.

However, if peak 15-min pedestrian volumes are available, the highest 15-min volume can be used directly without the application of a PHF.

Next, the peak 15-min flow is converted into a unit flow rate (pedestrians per minute per foot of effective path width):

$$v_p = \frac{v_{15}}{15 \times W_E}$$

Equation 24-3

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.

Step 3: Calculate Average Pedestrian Space

$Space = \frac{1}{Density}$

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. Pedestrian space is related to pedestrian speed and unit flow rate:

$$A_p = \frac{S_p}{v_p}$$

Equation 24-4

where

A_p = pedestrian space (ft²/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 24-1 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 in Section 4. The walkway LOS values can also be adapted to pedestrian plazas and pedestrian zones (exclusive pedestrian streets), as discussed in Section 4.

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 differentiates platooning from 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 during the analysis period, Exhibit 24-2 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 notes to Exhibit 24-1 and Exhibit 24-2, the LOS E threshold (i.e., capacity) in cross-flow situations occurs at a lower density (higher average space) 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 24-3. 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: 15 p/min/ft in the ascending direction.

PEDESTRIANS ON SHARED-USE PATHS

LOS for pedestrians on shared-use off-street paths 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 24-10 shows the steps taken to determine pedestrian LOS on shared-use paths.

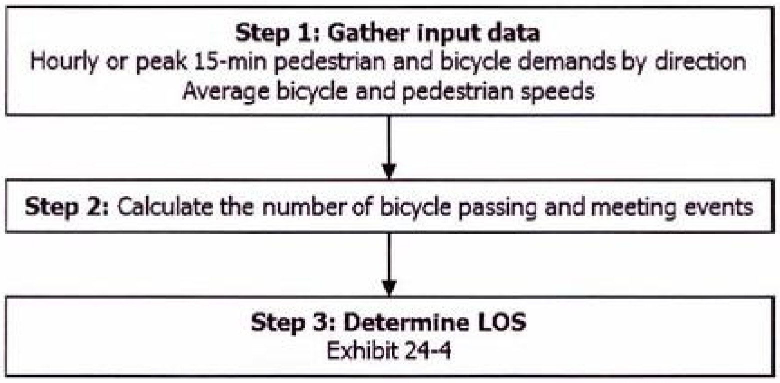


Exhibit 24-10
Flowchart for Analysis of Pedestrian LOS on Shared-Use Paths

Shared-use paths typically are open to users of nonmotorized modes such as bicyclists, skateboarders, and wheelchair users. They 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 or peak 15-min pedestrian and bicycle demands by direction, and
- Average pedestrian and bicycle speeds.

Step 2: Calculate Number of Bicycle Passing and Meeting Events

LOS is based on the overtaking of pedestrians by bicyclists. Passing occurs in the same direction; meeting occurs from the opposite direction. Pedestrian-to-pedestrian interaction is typically negligible.

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 numbers of passing and meeting events per hour are calculated by Equation 24-5 and Equation 24-6, respectively. 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 24-5

$$F_p = \frac{Q_{sb}}{PHF}\left(1 - \frac{S_p}{S_b}\right)$$

Equation 24-6

$$F_m = \frac{Q_{ob}}{PHF}\left(1 + \frac{S_p}{S_b}\right)$$

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).

If peak 15-min volumes by direction are known, they should be substituted for the Q_{sb}/PHF and Q_{ob}/PHF terms in the above equations. If only two-directional volumes are known, a directional distribution factor can be applied to the two-directional volume to estimate the directional volumes. (However, as mentioned previously, the LOS results are highly sensitive to the choice of directional factor, and field measurement of the directional distribution is recommended when possible.)

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 theory (*14*). When 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 PHF must be applied to convert them to the equivalent demand based on peak 15-min conditions. The total number of events is

Equation 24-7

$$F = \left(F_p + 0.5F_m\right)$$

where F is the total number of events on the path in events per hour, and the other variables are as defined previously.

Step 3: Determine LOS

Exhibit 24-4 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 off-street bicycle facility analysis is based on several factors that affect user perceptions.

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 (5) 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 bicyclists meet or pass other path users, and
- The bicyclist's freedom to maneuver.

On exclusive off-street bicycle facilities the number of passings and meetings is determined solely by the bicycle volume.

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 BLOS: 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.

BLOS 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 factors. These factors are combined into a single BLOS score. LOS thresholds relate to a specific range of LOS score values. Exhibit 24-11 shows the steps taken to determine the LOS of off-street bicycle facilities.

The following sections describe the steps for calculating BLOS 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 or peak 15-min 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).

Exhibit 24-11
Flowchart for Analysis of BLOS on Off-Street Facilities

Step 1: Gather input data
For each modal user (pedestrians, bicyclists, inline skaters, runners, and child bicyclists):
Hourly or peak 15-min 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 effective lanes

↓

Step 5: Calculate the probability of delayed passing

↓

Step 6: Calculate delayed passings per minute

↓

Step 7: Determine BLOS
Exhibit 24-5

↓

Step 8: Adjust LOS for low-volume paths

If peak 15-min directional volumes are known for each user group, the analysis can proceed directly to Step 2. Otherwise, the hourly directional flow rate on the path is calculated for each of the five modes on the basis of the hourly directional demand for the path and the path mode split:

Equation 24-8

$$q_i = \frac{Q_T \times p_i}{PHF}$$

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 (decimal), and

PHF = peak hour factor.

If only two-directional total path volumes are known, a directional distribution factor can be applied to the two-directional volume to estimate the directional volumes prior to entering them in Equation 24-8. (As mentioned above, LOS results are highly sensitive to the choice of directional factor, and field measurement of the directional distribution is recommended when possible.)

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. 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, 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

$$P(v_i) = P\left[v_i < U\left(1 - \frac{x}{L}\right)\right]$$

Equation 24-9

where

$P(v_i)$ = probability of passing user of mode i,

U = speed of average bicyclist (mi/h),

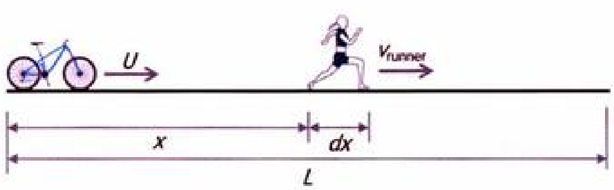
v_i = speed of a given path user of mode i (mi/h),

L = length of path segment (mi), and

x = distance from average bicyclist to user (mi).

Exhibit 24-12 provides a schematic of active passing events.

Exhibit 24-12
Schematic of Active Passing Events



Source: Adapted from Hummer et al. (5).

Because v_i is distributed normally, the probability in Equation 24-9 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 24-10

where $F(x)$ is the cumulative probability of a normal distribution of speeds with mean μ and standard deviation σ, 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 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:

Equation 24-11

$$A_i = \sum_{j=1}^{n} P(v_i) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

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 (5) has found that setting dx equal to 0.01 mi is appropriate for the purposes of the calculations shown in Equation 24-11 and below.

Equation 24-11 provides expected active passings by the average bicyclist for mode i. To determine total active passings of all modes, Equation 24-11 must be repeated for each individual mode and then summed:

Equation 24-12

$$A_T = \sum_i A_i$$

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 no user enters or exits the path at an intermediate point:

Equation 24-13

$$M_1 = \frac{U}{60} \sum_i \frac{q_i}{\mu_i}$$

where M_1 is the meetings per minute of users already on the path segment, and the other variables are as previously defined.

In addition to users already on the path segment, users who have yet to enter the segment will meet the average bicyclist within the segment. The probability of this occurrence is

Equation 24-14

$$P(v_{O,i}) = P\left(v_i > X\frac{U}{L}\right)$$

where

$P(v_{O,i})$ = probability of meeting opposing user of mode i,

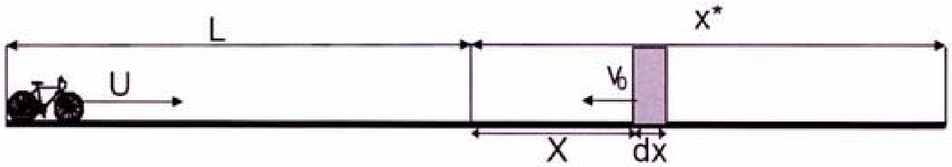
v_i = speed of path user of mode i (mi/h),

X = distance of user beyond end of path segment (mi), and

U = speed of average bicyclist (mi/h).

Because $v_{O,i}$ is distributed normally, the probability in Equation 24-14 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 applying Equation 24-10, substituting X for x. 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 (*5*). Exhibit 24-13 provides a schematic of meeting events.

Exhibit 24-13
Schematic of Meeting Events



Source: Adapted from Hummer et al. (*5*).

Similar to the process for calculating number of active passings (Equation 24-11), 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 24-15

$$M_{2,i} = \sum_{j=1}^{n} P(v_{O,i}) \times \frac{q_i}{\mu_i} \times \frac{1}{t} dx_j$$

where $M_{2,i}$ 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 $M_{2,i}$ across all mode groups:

Equation 24-16

$$M_T = \left(M_1 + \sum_i M_{2,i} \right)$$

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 Effective 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 24-14.

Exhibit 24-14
Effective Lanes by Path Width

Path Width (ft)	Effective Lanes
8.0–10.5	2
11.0–14.5	3
15.0–20.0	4

Source: Hummer et al. (*5*).

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 bicyclists require to pass other user modes are shown in Exhibit 24-15.

Exhibit 24-15 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. (*5*).

With the values in Exhibit 24-15, the probability that a given passing 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 $P_{n,i}$ of observing a blocked passing section for mode i is

Equation 24-17

$$P_{n,i} = 1 - e^{-p_i k_i}$$

where

$P_{n,i}$ = 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) = q_i/μ_i.

Equation 24-17 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, blocking a single lane in each direction; 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 24-18

$$P_{ds} = P_{no}P_{ns} + P_{no}(1 - P_{ns})(1 - P_{do})$$

Equation 24-19

$$P_{do} = P_{no}P_{ns} + P_{ns}(1 - P_{no})(1 - P_{ds})$$

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 24-18 and Equation 24-19 for P_{ds} results in

$$P_{ds} = \frac{P_{no}P_{ns} + P_{no}(1 - P_{ns})^2}{1 - P_{no}P_{ns}(1 - P_{no})(1 - P_{ns})}$$

Equation 24-20

Because P_{no} and P_{ns} are calculated from Equation 24-17, Equation 24-20 can be readily solved for P_{ds}.

Three-Lane Paths

Because a greater variety of possible scenarios may occur, the operations of three-lane paths are more complicated than those of two-lane paths. 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 blocks the rightmost lane in conjunction with opposing traffic occupying the other two lanes, or (*b*) side-by-side users block 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 24-21

$$P_{do} = P_{no}[P_{bs} + P_{ns}(1 - P_{ds})] + P_{bo}$$

Equation 24-22

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 24-21 and Equation 24-22 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 24-23

Substituting Equation 24-23 into Equation 24-21 results in

$$P_{ds} = \left[P_{ns}\left(P_{bo} + P_{no}(1 + D)\right) + P_{bs}\right]/(1 + P_{ns}P_{no})$$

Equation 24-24

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 (5) in which video data of more than 4,000 path users on U.S. shared-use paths were observed. Exhibit 24-16 shows the blocking frequencies by mode.

Exhibit 24-16
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. (*5*).

Therefore, $P_{bo,i}$ and $P_{bs,i}$, 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 24-16) by the probability that a user of mode i will be encountered, which is given by Equation 24-17. This process results in

Equation 24-25

$$P_{bs,i} = F_i \times P_{ns,i}$$

Equation 24-26

$$P_{bo,i} = F_i \times P_{no,i}$$

where F_i is the frequency with which mode i will block two lanes (from Exhibit 24-16), and all other variables are as previously defined. The probability that a user of any mode will block two lanes is thus given by

Equation 24-27

$$P_{bs} = \sum_i P_{bs,i}$$

Equation 24-28

$$P_{bo} = \sum_i P_{bo,i}$$

The probabilities that only a single lane will be blocked by a user of a given mode i, $P_{qs,i}$ and $P_{qo,i}$, are thus derived from the probability that at least one lane will be blocked (from Equation 24-17) minus the probability that two lanes will be blocked (from Equation 24-25 and Equation 24-26). These probabilities are

Equation 24-29

$$P_{ns,i} = 1 - e^{p_i k_{s,i}} - P_{bs,i}$$

Equation 24-30

$$P_{no,i} = 1 - e^{p_i k_{o,i}} - P_{bo,i}$$

where $k_{s,i}$ and $k_{o,i}$ 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 24-31

$$P_{ns} = \sum_i P_{ns,i}$$

Equation 24-32

$$P_{no} = \sum_i P_{no,i}$$

The values of P_{bs} and P_{bo} from Equation 24-27 and Equation 24-28 and the values of P_{ns} and P_{no} from Equation 24-31 and Equation 24-32 can now be substituted into Equation 24-23 and Equation 24-24 to determine the probability of delayed passing, P_{ds}. Because this delayed passing factor was calibrated by using peak hour volumes rather than peak 15-min volumes, a PHF is applied to convert A_T from peak 15-min flow rate conditions to hourly conditions.

Four-Lane Paths

On four-lane paths, the methodology assumes the path operates similarly to a divided four-lane highway, such that the probability of delayed passing is independent of opposing users, as 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 24-25 and Equation 24-27.

Step 6: Calculate Delayed Passings per Minute

The probability of delayed passing P_{ds} described above 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. Because there are five modes, five times five (25) modal pairs require calculation. The total probability of delayed passing is found by using

$$P_{Tds} = 1 - \prod_{m} (1 - P_{m,ds})$$

Equation 24-33

where P_{Tds} is the total probability of delayed passing, and $P_{m,ds}$ is the probability of delayed passing for mode pair m. The operator Π in Equation 24-33 indicates the product of a series of variables.

Finally, delayed passings per minute DP_m are simply the active passings per minute A_T multiplied by the total probability of delayed passing P_{Tds}:

$$DP_m = A_T \times P_{Tds} \times PHF$$

Equation 24-34

Because the DP_m factor was calibrated from peak hour volumes rather than peak 15-min volumes, a PHF is applied to convert A_T from peak 15-min flow rate conditions to hourly conditions.

Step 7: Determine BLOS

The BLOS score (Equation 24-35) 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 (5). The LOS C–D threshold represents the midpoint of the response scale used in the survey.

$$BLOS = 5.446 - 0.00809E - 15.86RW - 0.287CL - DP$$

Equation 24-35

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 [DP_m × 1.5/(180/60), 1.5] = min [DP_m × 0.5, 1.5].

The delayed passings factor DP is calibrated (*a*) to fall within the range of delayed passings (1–180 delayed passings per hour) observed during the research that developed this factor and (*b*) to produce a maximum change of three letters in the LOS result (5).

With the exception of the special cases discussed in Step 8, the bicyclist perception index is used directly with Exhibit 24-5 to determine bicyclist LOS on off-street facilities. As 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

It is not possible to achieve LOS A or B for narrow (e.g., 8-ft) paths by using Equation 24-35. Because 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 more than five to 10 weighted events per minute are assigned LOS B, unless Equation 24-35 would result in LOS A.

4. EXTENSIONS TO THE METHODOLOGIES

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 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, the circulation and amenity functions of a plaza sometimes conflict, as 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 24-17.

Exhibit 24-17
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 or color 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 seating areas and other plaza functions are added to the circulation space to determine the total plaza space required for pedestrian circulation and amenities.

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.

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 24-9, such as sidewalk café tables, are taken into account. Alternative performance measures may be considered that assess the street's attractiveness to pedestrians, because 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.

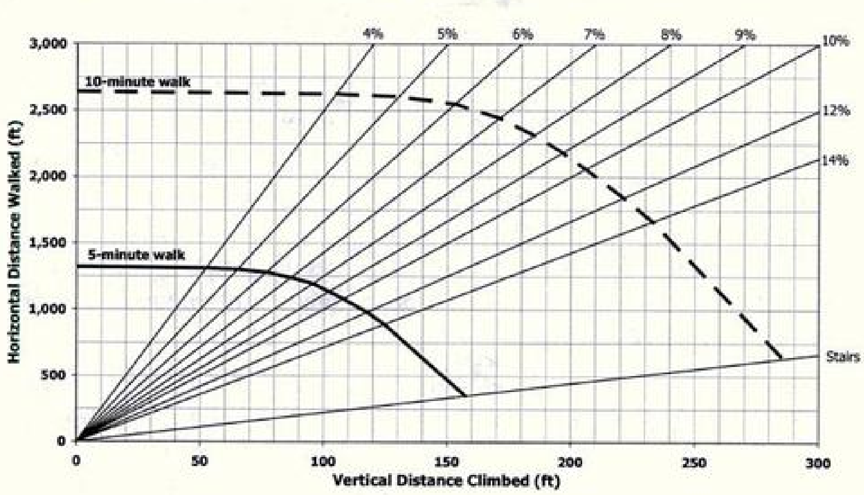
The HCM methodology is not suitable for pedestrian zones during times when delivery vehicles are allowed to use the street. The HCM methodology is also not applicable to the analysis of a low-speed street (e.g., a Dutch-style *woonerf*) shared by pedestrians, bicycles, and automobiles.

WALKWAYS WITH GRADES ABOVE 5%

Consult the latest version of the ADA Accessibility Guidelines *for guidance on the maximum slope allowed on an accessible route.*

Research (*9–11*) has shown no appreciable impact on pedestrian speed for grades up to 5%. As shown in Exhibit 24-18, 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, because the reduction in pedestrian speed is offset by closer pedestrian spacing (*9*). The stairway LOS table (Exhibit 24-3) would provide a conservative estimate of pedestrian LOS on steeper walkways.

Exhibit 24-18
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. When field observation on the path (or similar paths in the same region) indicates path users generally comply with the regulations, up to all the bicycle–pedestrian passing events could be converted to meeting events in proportion to the path users' compliance rate; this would result 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.

Where sufficient physical segregation of bicyclists and pedestrians occurs, it may be appropriate to treat the path as two separate facilities.

5. APPLICATIONS

EXAMPLE PROBLEMS

Section 2 of Chapter 35, Pedestrians and Bicycles: Supplemental, provides two example problems that illustrate the application of the off-street pedestrian and bicycle facility methods:

1. Comparison of pedestrian LOS on shared-use and exclusive paths, and
2. BLOS on a shared-use path.

EXAMPLE RESULTS

This section presents the results of applying this chapter's methods in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs, as well as to help evaluate whether their analysis results are reasonable. The exhibits in this section are not intended to substitute for an actual analysis and are deliberately provided in a format large enough to depict general trends in the results—but not large enough to pull out specific results.

Exclusive Off-Street Pedestrian Facilities

Exhibit 24-19 presents illustrative results showing how average pedestrian space relates to the combination of two-directional pedestrian demand volume and (*a*) effective path width or (*b*) average pedestrian speed. It can be seen that average pedestrian space increases with both increasing effective path width and increasing average pedestrian speed.

Exhibit 24-19
Illustrative Effect of Pedestrian Volume, Effective Path Width, and Average Pedestrian Speed on Average Pedestrian Space

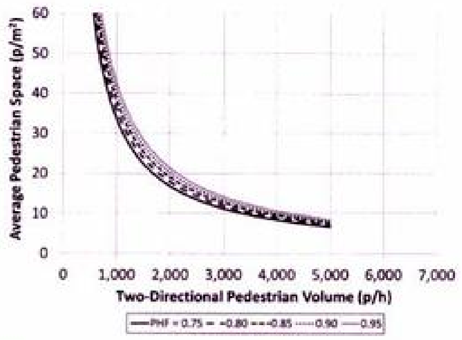
(a) Effective Path Width

(b) Average Pedestrian Speed

Note: Calculated using this chapter's methods, using PHF = 0.85. In Exhibit 24-19(a), average pedestrian speed = 3.4 mi/h. In Exhibit 24-19(b), effective path width = 10 ft.

Exhibit 24-20 presents illustrative results demonstrating how the calculated average pedestrian space during the peak 15 min (alternatively, the maximum hourly pedestrian volume that achieves a particular pedestrian space) relates to the choice of PHF. It can be seen that average pedestrian space is moderately sensitive to the choice of PHF.

Exhibit 24-20
Illustrative Effect of Pedestrian Volume and PHF on Average Pedestrian Space

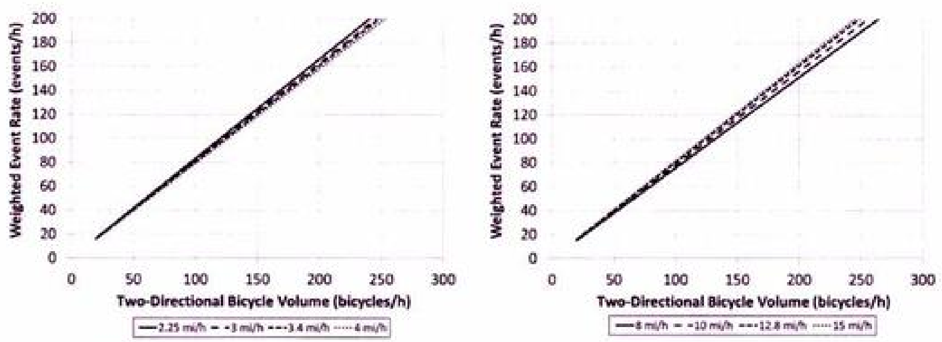


Note: Calculated using this chapter's methods, using an average pedestrian speed of 3.4 mi/h and an effective path width of 10 ft.

Pedestrians on Shared-Use Paths

Exhibit 24-21 presents illustrative results showing how the weighted event rate (i.e., weighted number of bicycle–pedestrian passings and meetings per hour) relates to the combination of two-directional bicycle demand volume and (*a*) average pedestrian speed or (*b*) average bicycle speed. It can be seen that the weighted event rate is relatively insensitive to path user speed and that even relatively low bicycle volumes produce diminished pedestrian quality of service (e.g., LOS D or worse, equivalent to 103 events per hour or more).

Exhibit 24-21
Illustrative Effect of Bicycle Volume, Average Pedestrian Speed, and Average Bicycle Speed on Weighted Event Rate



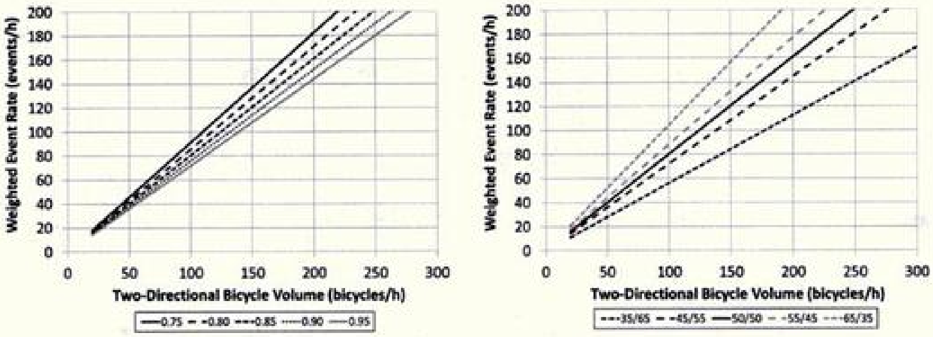
(a) Average Pedestrian Speed

(b) Average Bicycle Speed

Note: Calculated using this chapter's methods, using PHF = 0.85 and a 50/50 bicycle directional split. In Exhibit 24-21(a), the average bicycle speed is 12.8 mi/h. In Exhibit 24-21(b), the average pedestrian speed is 3.4 mi/h.

Exhibit 24-22 presents illustrative results demonstrating how the weighted event rate varies with (*a*) PHF and (*b*) the directional distribution of passing bicyclists. The weighted event rate is moderately sensitive to PHF and highly sensitive to the directional distribution. A directional imbalance with more passing bicyclists has a greater impact than an imbalance with more meeting bicyclists; note that Equation 24-7 weights a meeting event as equivalent to half a passing event. The directional distribution can be relevant to shared-use paths used as bicycle commuter routes, as well as recreational trails where bicyclists travel out and back via the same route, and thus a greater proportion of bicyclists travel in a given direction at a given time of day.

Exhibit 24-22
Illustrative Effect of Bicycle Volume, PHF, and Directional Distribution of Passing Bicyclists on Weighted Event Rate



(a) PHF

(b) Distribution of Passing Bicyclists

Note: Calculated using this chapter's methods, using an average pedestrian speed of 3.4 mi/h and an average bicycle speed of 12.8 mi/h. In Exhibit 24-22(a), the directional distribution is 50/50. In Exhibit 24-22(b), the PHF is 0.85.

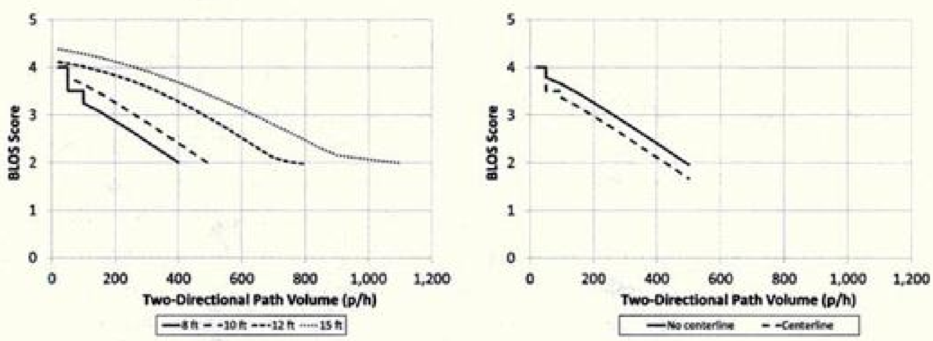
Off-Street Bicycle Facilities

Exhibit 24-23 presents illustrative results showing how the BLOS score relates to the combination of two-directional path volume and (*a*) path width or (*b*) centerline width.

It can be seen in Exhibit 24-23(a) that the BLOS score improves substantially as path width increases, which allows bicyclists to maneuver more freely. On the two-lane (i.e., 8- and 10-ft) paths, the effect of the LOS adjustment for low-volume paths (Step 8 of the methodology) can be seen as stair steps in the curves, as the LOS is automatically set to LOS A or LOS B under low-volume conditions. On the three- and four-lane (i.e., 12- and 15-ft) paths, an inflection point can be seen in the curve at higher path volumes. This inflection point is an effect of the delayed passings (*DP*) variable in Equation 24-35, which caps the delayed passing rate at 1.5 delayed passings per minute. Once this point is reached, the BLOS score declines much more slowly.

Exhibit 24-23(b) shows that the BLOS score on paths with centerlines is always 0.287 lower than on paths without centerlines, corresponding to the coefficient for this factor in Equation 24-35, except under low-volume conditions, when the Step 8 LOS adjustment applies.

Exhibit 24-23
Illustrative Effect of Path Volume, Path Width, and Centerline Presence on BLOS Score



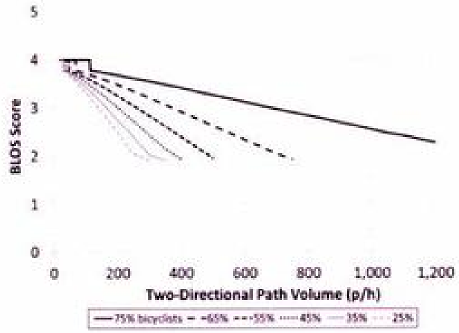
(a) Path Width

(b) Centerline Presence

Note: Calculated using this chapter's methods, using the default mode splits and modal user speeds given in Exhibit 24-6, a PHF of 0.85, and a 50/50 path user directional distribution. In Exhibit 24-23(a), no centerline is present. In Exhibit 24-23(b), the path width is 10 ft.

Exhibit 24-24 presents illustrative results demonstrating how the BLOS score varies with the percentage of path users who are bicyclists. It can be seen that the BLOS score varies widely for a given path volume depending on the percentage of bicyclists (alternatively, the maximum path volume that achieves a given LOS varies greatly depending on the percentage of bicyclists). The relative percentages of other path users also affect the BLOS score, with the slowest modal users (e.g., pedestrians, child bicyclists) having the biggest effect. However, accurately estimating the bicyclist percentage is more important, in terms of the impact on the final result, than accurately estimating the relative proportions of the other path user types.

Exhibit 24-24
Illustrative Effect of Path Volume and Bicyclist Percentage on BLOS Score



Note: Calculated using this chapter's methods with the default modal user speeds given in Exhibit 24-6, a PHF of 0.85, and a 50/50 path user directional distribution. The mode splits for runners, inline skaters, and child bicyclists are as given in Exhibit 24-6, with pedestrians making up the balance of the path users.

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 a 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 24-1 or Exhibit 24-2. The effective width would be computed by solving the pedestrian unit flow-rate equation backward. To avoid pedestrian spillover (i.e., where pedestrians walk outside the path to pass other users), 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 24-5. By holding all but one path-user group's demand constant and solving the events equation backward (e.g.,

13. Virkler, M. Quality of Flow Along Pedestrian Arterials. Presented at 18th Annual Australian Road Research Board Transport Research Conference–Transit New Zealand Land Transport Symposium, Christchurch, New Zealand, Sept. 1996.

14. Botma, H. Method to Determine Level of Service for Bicycle Paths and Pedestrian–Bicycle Paths. In *Transportation Research Record 1502*, Transportation Research Board, National Research Council, Washington, D.C., 1995, pp. 38–44.

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.

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