Highway Capacity Manual Reference Guide

November 2022

U.S. Department of Transportation
Federal Highway Administration

FOREWORD

Welcome to the Federal Highway Administration (FHWA) Highway Capacity Manual (HCM) Reference Guide (HCMRG). This document provides sufficient background knowledge and guidance for professionals conducting studies or reviewing analyses based on the HCM methodology to assess whether they meet the objectives of each project or study being conducted. This Guide covers the analysis and review of all methodological chapters, some of which are quite complex and contain many computations that can be misunderstood – which is where the guidance becomes most beneficial.

The technical approach within the HCMRG was intentionally not to repeat the HCM procedures themselves but to provide key insights into critical parameters and their effects on results that would be especially useful in reviews.

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The objective of the Highway Capacity Manual Reference Guide (HCMRG) is to provide simple explanations and applicable guidance for use in typical highway capacity analysis tasks. This Guide covers the analysis and review contained within all Highway Capacity Manual (HCM) 7th Edition methodological chapters, some of which are quite complex and contain many computations that can be misleading or misunderstood—which is where the guidance in the HCMRG becomes most beneficial. The technical approach intentionally taken within the HCMRG was not to repeat the HCM procedures, but to provide key insights into critical parameters and their effects on results that would be especially useful in reviews. The HCMRG is organized by HCM 7th Edition chapters with specific references included with each topic.</td></tr>
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Acronyms

AP	analysis period
ATDM	active transportation demand management
AWSC	all-way stop control
BFFS	base free-flow speed
CAF	capacity adjustment factor
CAV	connected and automated vehicles
CFI	continuous flow intersection
DDI	diverging diamond interchange
DLT	displaced left turn
DCD	double crossover diamond
EDTT	extra distance travel time
ETT	experienced travel time
FD	follower density
FFS	free-flow speed
FYA	flashing yellow arrow
g/C	green-to-cycle ratio
GP	general purpose
HCM	Highway Capacity Manual
HCMRG	Highway Capacity Manual Reference Guide
IQA	incremental queue accumulation
ITS	intelligent transportation systems
LOS	level of service
ML	managed lane
MOE	measure of effectiveness
MUT	median U-turn
OD	origin-destination
PCE	passenger car equivalent
pc/h	passenger cars per hour
pc/hg/pl	passenger cars per hour green per lane
pc/mi/ln	passenger cars per mile per lane
PHF	peak hour factor
PPEAG	Planning and Preliminary Engineering Applications Guide
PTTI	planning time index
RCUT	restricted crossover U-turn
RRFB	rectangular rapid flashing beacon
RTOR	right turn on red
s/veh	seconds per vehicle
SAF	speed adjustment factor
SUT	single-unit truck
TT	tractor-trailer
TTI	travel time index
TWLTL	two-way left-turn lane

TWSC	two-way stop control
v/c	volume-to-capacity ratio
va/c	ratio of volume served and capacity
VR	volume ratio
VDH	vehicle hours of delay

Symbols

$f_{c,x,y}$	conflicting flow factors
L_{A1}	auxiliary lane addition 1.
L_{A2}	auxiliary lane addition 2.
L_B	base length
L_{CW}	road length
$L_{CW\text{-}MAX}$	maximum road length
$L_{CW\text{-}MIN}$	minimum road length.
L_{EQ}	equilibrium distance
L_{MAX}	maximum weaving length
L_S	short length
N_{WL}	minimum weaving maneuver lanes
P_{FM} or P_{FD}	proportion of (merging or diverging) vehicles in lanes 1 and 2
S	average travel speed
V	hourly volume
V_{15}	volume during the peak 15 min of the analysis hour
$v_{c,x}$	conflicting flow rate for movement x
V_F	freeway volume
V_{FF}	freeway-to-freeway demand flow rate in the weaving segment
V_{FR}	freeway-to-ramp demand flow rate in the weaving segment
V_R	ramp volume
V_{RF}	ramp-to-freeway demand flow rate in the weaving segment
V_{RR}	ramp-to-ramp demand flow rate in the weaving segment
W	weaving intensity factor

CHAPTER 1. INTRODUCTION

The *Highway Capacity Manual, 7th Edition: A Guide for Multimodal Mobility,* was released in early 2022, incorporating the latest research on highway capacity, level of service (LOS), and multimodal analysis to keep pace with the needs of its users and society. The HCM is "the fundamental reference on concepts, performance measures, and analysis techniques for evaluating the multimodal operation of streets, highways, freeways, and off-street pathways." (HCM p. 10-25)

STRUCTURE AND NEWS ON THE HCM 7TH EDITION

Overall, in the 7th Edition of the HCM, more than 350 pages are updated from the 6th Edition. The most notable changes in the 7th Edition of the HCM include:

- Guidance on the application of HCM methods to determine capacity impacts of connected and automated vehicles (CAVs).
- A new network analysis method to evaluate spillback between freeways and urban streets, estimate travel time across facilities, and conduct lane-by-lane analysis for freeways.
- A new two-lane highway analysis method offers improved analysis of two-lane highway capacity and operational performance.
- Enhancements to existing pedestrian analysis methods at signalized intersections and uncontrolled crossings.

The 7th Edition of the HCM is organized into 38 chapters within 4 volumes:

I. Concepts

II. Uninterrupted Flow

III. Interrupted Flow

IV. Applications Guide (available online)

The major applications of the HCM are to:

- Define performance measures and describe survey methods for key traffic characteristics,
- Provide methodologies for estimating and predicting performance measures, and
- Explain methodologies at a level of detail that allows readers to understand the factors affecting multimodal operation.

The HCM is prepared for use primarily by engineers and analysts who work in the field of *traffic operations*, *planning*, or *highway geometric design*. To use the manual effectively and to apply its methodologies, some technical background is desirable, typically in the form of university-level training or technical work in a public agency or consulting firm.

However, the HCM explains the methodologies at a certain level of detail, allowing readers to understand the factors affecting multimodal operation. Each method in HCM produces performance measures that can be understood and are applicable to a broader range of decisionmakers and professionals, including planners, management personnel, educators, air quality specialists, noise specialists, elected officials, regional land-use planners, and interest groups representing particular users.

Objective of the *Highway Capacity Manual Reference Guide* (HCMRG)

The objective of the HCMRG is to provide sufficient background knowledge and guidance for agencies and professionals analyzing or reviewing analyses based on the HCM methodology to assess whether the analyses meet the objectives of each project or study being conducted. The HCMRG is intended to supplement, add to, fill gaps, further explain, and provide key insights into aspects of some complex procedures that are often misunderstood and incorrectly applied.

The following are examples of challenges to the HCM user with references to the pages within this HCMRG that provide more detailed guidance:

- Multiple-Period Analysis: With appropriate arrival demand data, quantifying unmet demand for each period as initial queues for subsequent periods throughout the congested analysis is critical to generate an accurate delay estimation for oversaturated conditions that can be underestimated otherwise. (pp. 7–9, 48–49)

- Queue Spillover: Using alternative tools to compute the additional delay to adjacent lanes when turning-lane queues exceed the available storage is essential to overcoming this HCM limitation, which ignores queue spillover interference with other lane groups. (pp. 12–14, 57)

- Base Saturation Flow Calibration: When analyzing signalized intersections in larger cities and smaller towns, the default values of 1,900 passenger cars per hour green per lane (pc/hg/pl) and 1,750 pc/hg/pl should be overridden with locally calibrated values that can range from 1,400 pc/hg/pl to 2,100 pc/hg/pl. (p. 50)

- Access Point Interaction: Vehicle platoons from upstream signals can be slowed and fragmented by midblock access points on urban streets, affecting the downstream arrival times severely, so these must be included in the analysis to calculate accurate proportions arriving on green and how to assess the effect of that value on uniform delay. (p.60).

- Simulation Compliance: When using stochastic simulation tools as alternatives to deterministic HCM procedures, underlying parameters must be adjusted to match those in the HCM by following the guidance for using alternative tools throughout the HCM. (pp. 12–14)

- Systemwide Oversaturation: Beginning and ending unsaturated in time and distance by requiring that the first and last segments are unsaturated for every period and that all segments are unsaturated for the first and last period to constrain congestion within the analysis frame. Also, the HCM 7th Edition introduces new methods for evaluating interactions between freeways and arterials. (pp. 10, 36, 50, 85–87)

- Managed Lane (ML) Cross-Weave Effects: Reduces capacity for freeway weave, merge, and diverge segments. This reduction in capacity for vehicles moving to and from on- and off-ramps to and from MLs must be considered when making the required capacity checks for freeway segments. For design-level analysis, the location of ML access points becomes a critical decision, as it will affect the cross-weave length. (pp. 26–27)

HCMRG Structure

The following section of this document addresses general traffic analysis concepts that are needed across the several methods of the HCM. Subsequent sections explain the main concepts and variables to be considered by the analyst or reviewer, focusing on key aspects that will affect the results the most, and the resulting performance measures for each HCM chapter. At the end of each section, a table is provided to serve as a checklist that the analyst or reviewer should look for in the HCM analysis. The checklist table summarizes variables and concepts discussed in the text and their type, categorized in:

- Modeling: Key modeling aspects of the analysis (i.e., freeway segment type).
- Input: Main inputs of each method (i.e., demand, number of lanes).
- Calibration: Calibration parameters with a major impact on the analysis (i.e., saturation flow, capacity adjustments).
- Measures of effectiveness (MOE): MOE and outputs of the model. Key MOEs and service measures are in bold (i.e., traffic density, delays, queues).

For inputs and some calibrated parameters, HCM default variables are provided in the table. The defaults serve the purpose of serving as a reference and should only be used when field data cannot be obtained. The reviewer should assess whether default value use is adequate in each case.

Note that references specific to pages, equations, and exhibits in the HCM will appear in parentheses following the discussion of issues related to those items. These parenthetical contents are intended to better aid the reader in tracking between this HCMRG and the relevant content within the HCM 7th Edition.

CHAPTER 2. GENERAL CONCEPTS

FACILITY TYPES

The HCM includes core and supplemental chapters presenting methodologies covering isolated elements or facility-wide analysis for:

- Freeway Facilities:
  - Chapters 10 and 25 – Freeway Facilities Core Methodology
  - Chapter 11 – Freeway Reliability Analysis
  - Chapters 12 and 26 – Basic Freeway Segments
  - Chapters 13 and 27 – Freeway Weaving Segments
  - Chapters 14 and 28 – Freeway Merge and Diverge Segments
- Highways
  - Chapters 12 and 26 – Multilane Highways
  - Chapters 15 and 26 – Two-Lane Highways
- Unsignalized Intersections:
  - Chapters 20 and 32 – Two-Way Stop-Controlled Intersections
  - Chapters 21 and 32 – All-Way Stop-Controlled Intersections
  - Chapters 22 and 33 – Roundabouts
- Urban Corridors Methodologies:
  - Chapters 16 and 29 – Urban Street Facilities
  - Chapters 18 and 30 – Urban Street Segments
  - Chapters 19 and 31 – Signalized Intersections
  - Chapters 23 and 34 – Ramp Terminals and Alternative Intersections
  - Chapters 17 and 37 – Urban Streets Reliability and Active Transportation Demand Management (ATDM)
- Freeway and Urban Streets Corridors Methodologies
  - Chapter 38 – Network Analysis

KEY PARAMETERS AND PERFORMANCE MEASURES

Demand Variables

Travel demand forecasting and planning-level studies typically use average annual daily traffic as a measure of demand for various purposes. Traffic engineering analysis using HCM methods requires a higher level of detailing to assess how different facilities will operate under different conditions. Therefore, hourly flow rates are required.

The Analysis Period (AP) is defined as the time interval evaluated by a single application of an HCM methodology. (HCM p. 9-2) Specifically, 15-min intervals within the study hour (usually peak periods) are the recommended time resolution for HCM analysis.

Whenever only hourly volumes are available, a peak hour factor (PHF) is used to adjust the volume to the peak 15-min flow rate within the study hour.

The PHF is computed as:

$$PHF = \frac{V}{4 \times V_{15}}$$

Where V is the hourly volume and V_{15} is the volume during the peak 15 min of the analysis hour.

When PHF is known, it can be used to convert a peak hour volume to a peak flow rate:

$$v = \frac{V}{PHF}$$

Where v is the flow rate for the 15-min peak period.

A PHF close to 1.0 means that traffic flow is fairly evenly distributed across the hour. If there is a major generator that has a defined open or close time, like an office complex, there may be a sharper peak of traffic flow within one of the 15-min intervals, and this would tend to reflect a PHF closer to 0.75 or even lower.

Adjusted flow rate is used to compute the volume-to-capacity (v/c) ratio used for calculating delay for interrupted flow procedures and density for uninterrupted flow methods, both of which are used to determine the LOS. Care must be taken to get the PHF correct by collecting traffic volumes in 15-minute increments so that the PHF is calculated directly for existing conditions. Extrapolating these values for use in analyses that involve traffic projections should be done in a logical way, recognizing that the PHF will generally rise as traffic levels increase, but starting with field data is vital. The PHF may range from 0.25 to 1.00, with typical default values from 0.88 to 0.95.

Whenever demand is higher than the section capacity, the difference between demand and volume served (throughput) needs to be acknowledged. Demand relates to the number of vehicles that would like to be served by a roadway element, while volume relates to the number that are actually served and may be constrained by the segment capacity. (HCM p. 8-3)

Most HCM methods also require that demand be differentiated between passenger vehicles and heavy vehicles. Therefore, it is vital that traffic data for HCM analysis are provided for automobiles, trucks, and buses. Pedestrian and bicycle counts can be used to refine motorized vehicle analysis and to perform a multimodal LOS analysis.

Level of Analysis

The HCM methodologies can be used for various levels of analysis and stages of a project with the overall objectives. They can provide a rich set of performance measures, including LOS, travel time, speed, and delay, among others. The HCM LOS analysis can be tailored for three general categories:

- Operations: Assess and improve the quality of service in which a facility or component operates in the analysis period.

- Design: Based on target quality standards, provide design elements specifications, including the number of lanes, lane geometry, and intersection control type.
- Planning and preliminary engineering: Provide high-level performance measures for use in a range of applications, including but not limited to preliminary facility and intersection design for further detailed study in later stages of a project, cost-benefit evaluation, and strategic planning.

Travel Modes

The HCM presents LOS methods for motorized vehicles across the entire manual. Multimodal methods are presented across several chapters for different facility types, including LOS for pedestrians, bicycles, and transit modes. Table 1 lists multimodal methods across HCM core chapters. In addition to those, "Chapter 24 – Off-Street Pedestrian and Bicycle Facilities" presents performance measures and methods for the named facilities, including non-motorized-traffic-only streets, paths, plazas, and other pedestrian zones.

Table 1. Multimodal methods across HCM chapters.

Chapter	Pedestrian	Bicycle	Transit
Chapter 12 – Multilane Highways		✓	
Chapter 15 – Two-Lane Highways		✓	
Chapter 16 – Urban Street Facilities	✓	✓	✓
Chapter 18 – Urban Street Segments	✓	✓	✓
Chapter 19 – Signalized Intersections	✓	✓	
Chapter 20 – Two-Way Stop-Controlled Intersections	✓		
Chapter 21 – All-Way Stop-Controlled Intersections	✓*		
Chapter 22 – Roundabouts	✓*	✓*	

*Partial guidance.

Single-Period versus Multiple-Period Analysis

A typical peak period analysis is intended to measure the worst operational conditions, usually taking the peak 15-min within the analysis hour as the analysis period.

The HCM offers multiple-period and peak-period analyses for freeways and urban streets. It is essential to understand where these analysis types should be used.

The operational condition can range from free-flow (LOS A) to near capacity (LOS E).

The single-period analysis cannot account for the effects of queue formation and dissipations over the facility in subsequent analysis periods, nor will it be able to evaluate the operations for the unserved demand within the peak period. As such, the "single-period" analysis should not be used for oversaturated conditions.

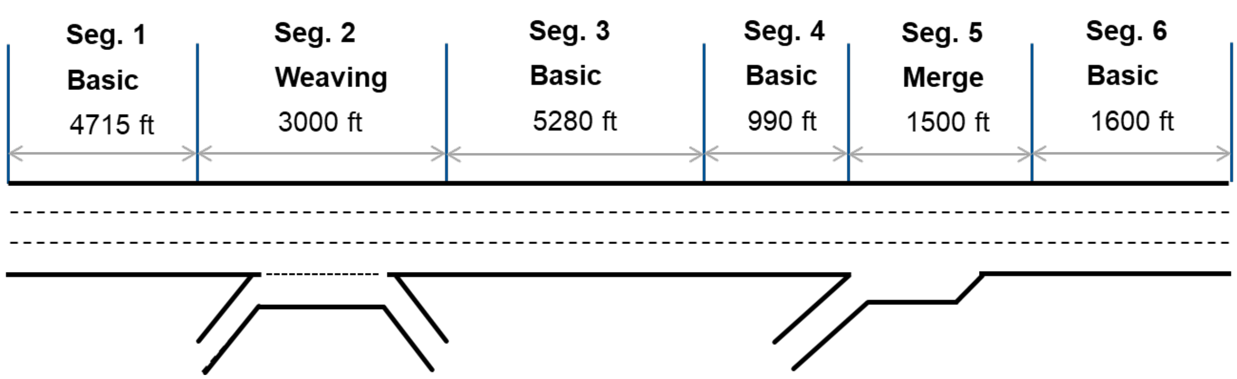
On the other hand, the multiple-period analysis focuses on several analysis periods, each being a single study period. Demand volumes are provided for all 15-min analysis periods. Thus, PHF is not used for this approach (or is set as 1.00 in most software applications). The major differences between the two approaches are summarized in table 2.

Table 2. Single-period versus multiple-period analysis.

Element	Peak-Period Analysis	Multiple-Period Analysis
Input demand	One value for the entire study	Provided at every 15 min AP
PHF	Calculated as a function of the busiest 15-min AP	Not needed (PHF = 1.0)
LOS output	One for the entire study period	One for each AP, considering the queue from the previous AP

When demand exceeds capacity (LOS F), the unserved demand will form queues that can last several 15-min APs before they get cleared. The demand served will be constrained by queue dissipation rates and bottleneck capacities and will always be less than the arrival demand for that single 15-min period.

The examples shown in table 4, table 5, and table 6 help to better understand the difference between the two approaches when the operational conditions during the peak period are oversaturated. In this freeway facility, the onset of congestion occurs in the 18:00–18:15 AP, with segment 5 (merge) being a bottleneck. The speed heatmap shows a freeway segment's average speed (mi/h) during the peak period. As seen, the darker color corresponds to slower speeds indicates the existence of queues:



Source:

Figure 1. Illustration. Single-period analysis.

Table 3. Onset of congestion.

Analysis Period	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6
18:00 – 18:15	**70.0**	**54.7**	**16.4**	**7.3**	**58.4**	**69.1**

Onset of congestion

As discussed, the single-period analysis cannot determine the operational impacts of oversaturated conditions in the subsequent analysis periods. Next, the same analysis is extended for a longer study period. Table 4 shows the speed heatmap for the same facility. In fact, the period after the onset of congestion will operate in the worst condition due to queue operations from unserved vehicles in the previous period. This condition is reflected by facility-wide performance measures, presented in table 5: vehicle-hours of delay (VHD), speed, density, travel time, and LOS.

Besides failing to determine the worst operational condition, the single-period analysis cannot fully assess the effects of congestion during the study period.

Table 4. Speed heatmap – multiple-period analysis.

	Speed (mi/h)						
Analysis Period	**Seg 1**	**Seg 2**	**Seg 3**	**Seg 4**	**Seg 5**	**Seg 6**	
17:30 – 17:45	69.9	63.5	69.8	69.8	67.5	69.8	
17:45 – 18:00	70.0	64.4	70.0	70.0	67.3	69.8	
18:00 – 18:15	70.0	54.7	16.4	7.3	58.4	69.1	Onset of congestion
18:15 – 18:30	70.0	8.6	7.0	7.0	65.4	69.6	Queue propagation
18:30 – 18:45	70.0	7.1	11.5	11.7	62.7	68.5	
18:45 – 19:00	70.0	61.0	24.3	24.1	57.6	57.7	Queue clearing
19:00 – 19:15	70.0	62.4	70.0	70.0	67.8	64.8	
19:15 – 19:30	70.0	62.4	70.0	70.0	68.1	65.8	

Table 5. Facility MOEs – multiple-period analysis.

Analysis Period	**VHD (veh-h/AP)**	**Speed (mi/h)**	**Density (pc/mi/ln)**	**Travel time (min)**	**LOS**	
17:30 – 17:45	1.4	68.5	18.1	2.9	C	
17:45 – 18:00	1.0	68.7	14.9	2.9	B	
18:00 – 18:15	54.6	29.2	36.2	6.9	F	Onset of congestion
18:15 – 18:30	150.4	14.2	71.5	14.2	F	Queue propagation
18:30 – 18:45	149.9	16.1	73.7	12.4	F	
18:45 – 19:00	48.5	37.6	40.4	5.3	E	Queue clearing
19:00 – 19:15	1.9	67.7	16.8	3.0	B	
19:15 – 19:30	1.8	67.9	16.1	3.0	B	

pc/mi/ln = passenger cars per mile per lane; veh-h/AP = vehicle hours per analysis period.

The same logic applies to urban street analysis. As such, the multiple-period analysis can account for the impact of queues while forming and dissipating, providing a comprehensive view of facilities and intersection operational conditions.

Both single-period and multiple-period analyses will report the same results when a facility or intersection is under capacity. However, when a traffic condition is oversaturated, single-period peak analysis will generate a misleading result. It is always advised to evaluate the operational condition of the analysis to determine if there are queues or not. Then, based on the result, pick the correct type of analysis. Of course, a multiple-period analysis can always be used to estimate performance measures and overcome the PHF usage dilemma, regardless of traffic conditions.

In summary, the recommended analysis approach is:

- For undersaturated conditions (LOS E or better), peak period analyses provide a reasonable estimation of performance with a small amount of required effort.
- For oversaturated conditions (LOS F), multiple-period analyses must be used to account for the effects of queueing on freeways.

Both HCM methods (freeway facilities and urban streets facilities) can analyze oversaturation conditions and model queues between multiple analysis periods.

HCM's first requirement for the congestion analysis is for the first and last 15-min AP conditions to be undersaturated; i.e., congestion should be contained within a study period consisting of multiple 15-minute analysis periods. The second condition is to for the first and last segment (either freeway or urban streets) to be uncongested. These requirements ensure that the impact of congestion can be fully included in the analysis.

Single-Point Analysis

The single point (e.g., single signalized intersection) and single segment (e.g., a single weave segment) analysis method can be used to assess individual components of a facility or system. This approach is ideal for planning or design-level applications to determine specific features to meet a target LOS. Common examples include the following:

- Defining the number of lanes on a freeway or highway segment.
- Defining ramp location and geometry to avoid or mitigate weaving operation issues.
- Conducting a planning-level comparison of different options for an isolated intersection.

For oversaturated conditions, a single-point analysis will simply assign a LOS of F and provide little information as to the impact of the congestion on upstream segments.

Service Measures and Other Performance Measures

The HCM provides a macroscopic set of methods to simulate the operational conditions of different facilities. The performance measure used to define the LOS is named "service measure." A vast number of other performance measures are also generated from the analyses. Key performance measures that HCM can estimate across different methods are summarized in table 6. Single-period or multiple-period analysis options are available for freeway and urban street facilities.

Table 6. HCM main performance measures and analysis scope.

Performance Measure	HCM Chapter				
	Freeways / Multilane Highways	Urban Streets	Signalized Intersections	Unsignalized Intersections	Two-Lane Highways
Traffic density	✓				Follower density
Average speed	✓	Service Measure			✓
Travel time	✓	✓			✓
Control delay			Service Measure	Service Measure	
Segment delay	Service Measure	✓			
Vehicle miles travelled (VMT)	✓[1]	✓			
Queues	✓	✓	✓	✓	
Travel time reliability	✓[2]	✓[2]			

[1]Similar performance measures also computed.
[2]Reliability measures include the mean travel time index (TTI), TTI percentiles, and level of travel time reliability.

Performance measures for signalized and unsignalized intersections are provided by movements associated with specific lane groups. In the HCM 7th Edition, "Chapter 38 – Network Analysis" includes methods to estimate performance measures for freeway facilities on a lane-by-lane basis.

While average control delay is used to determine the LOS in all intersection analyses, thresholds differ between signalized and unsignalized control types. This presents a dilemma when comparing the delay between these control types, for which the HCM provides no guidance. One option for the analyst is to compare delay directly or as part of a cost-benefit analysis.

SOFTWARE SELECTION

Key points to consider when selecting software to facilitate HCM analysis:

- HCM Compliance Level. Every traffic analysis software has a certain degree of compliance with the HCM methods. HCM-based MOEs, when reported in traffic analysis software, do not guarantee compliance with the HCM. HCM compliance requires not only the LOS value but also the corresponding service measured to be calculated following HCM principles and assumptions. As an example, a non-HCM method can be used to estimate delay at the intersections. HCM LOS criteria can be used to determine LOS; however, the estimated LOS is not an HCM-based LOS estimate unless delay is also measured following the same assumptions as in the HCM (compatible analysis periods, signal phasing assumption etc.). Caution needs to be used when using traffic analysis software to determine the degree of compliance with the HCM. The level of compliance with the HCM for each software package may first be assessed by comparing the software results with the HCM example problems provided by the manual in its supplemental chapters.

- Different traffic analysis software has different ways of collecting inputs. HCM provides a wide set of default values, but not all traffic analysis tools use HCM default values. When input is not entered, caution must be taken when determining which default values are used.

- When software is used to facilitate HCM analysis, the user must provide a summary of the assumptions and inputs along with the results report. The results are derived based on user and software assumptions in considering input values.

CAV METHODOLOGY

The 7th Edition of HCM provides capacity adjustments for a predefined mix of connected and automated vehicles (CAVs) in the traffic mix for freeways, roundabouts, and signalized intersections. It also provides daily and hourly maximum service volumes for basic freeway segments for different proportions of CAVs in the traffic stream.

Although CAVs are still a developing technology, transportation agencies have an immediate need as part of their long-range planning efforts to account for CAVs' potential ability to increase existing roadways' throughput. The combination of connectivity and automation can reduce reaction times and enable closer car-following distances, facilitating higher traffic densities and higher capacities. This combination may also improve travel time reliability by reducing crashes. Chapters 26, 31, and 33 provide guidance on adapting the HCM freeway, signalized intersection, and roundabouts methods to forecast traffic operations with CAVs present in the traffic stream.

The main application of CAV methodology in the HCM is for long-range planning analysis. These adjustments are not meant to be used for operations-level analysis (e.g., signal timing or ramp metering)

The basis of the CAV methodology for different facility types includes the following:

- *Freeways*: in the form of **Capacity Adjustment Factor** (CAF) for a basic, merge, diverge and weave segment types. A series of lookup tables are provided in HCM chapter 26 to define CAV CAFs for different freeway segment types when a certain percentage of CAVs are present in the traffic mix.

- *Signalized Intersections*: in the form of **Saturation Flow Rate Adjustment Factors**. A series of look-up tables are provided in HCM chapter 31 to define the CAV adjustment factors for through, protected, and permitted-left movement saturation flow rates when a certain percentage CAVs are present in the traffic mix.

- *Roundabouts*: in the form of **Adjustment Factors for the Parameters of the Capacity Model**. Look-up tables are provided in chapter 33 to define the roundabout capacity model's parameters when a certain percentage of CAVs are present in the traffic mix.

These CAV methodologies for different facilities rely on one input from the user: the proportion of CAVs in the traffic stream, also called CAV market penetration rate. This percentage will significantly affect the results for the scenarios with CAVs; therefore, care should be taken when making assumptions about this variable.

ANALYTICAL MODELS VERSUS SIMULATION

Microscopic traffic simulation is a comprehensive method for simulating how the traffic condition will be for a predetermined set of geometry, traffic demand, and control types. The simulation approach is stochastic and can present average performance measures and their variability. Even though microsimulation is comprehensive and capable of finding almost any output that analysts are looking for, it is time and data-intensive. In addition, a significant amount of time should be devoted to calibrating the microsimulation model before the results can be trusted.

In contrast, HCM's macroscopic models are deterministic and have been developed for typical geometry and traffic demand configurations. The resulting performance measures reflect a standardized average that can be representative of most cases. However, calibration for local conditions is desirable. For example, base saturation flow rates at signalized intersections and critical headways at stop-controlled intersections are extremely important in many situations. Also, HCM methods are not designed for some complex geometries. Generally, using HCM methods requires less time and data compared to the microsimulation option.

When to use HCM methods. HCM planning-level methods can serve as good screening tools to analyze the traffic's operational conditions. HCM is also used for operation, design, traffic impact, and other studies affecting multimillion-dollar decisions. When there is a lack of data, HCM's default data can be used to fill the gap and lead to a credible result. Strong consideration should be given to overriding defaults with defendable local information and knowledge, especially when defaults are not reasonable for a given analysis.

Also, HCM methods may be ideal when standardized models for comparison are desirable or required. Other cases where the HCM presents methods that can replace or supplement simulation include:

- Cases where models are needed to estimate free-flow speeds (FFS) on freeway and highway segments.
- Reliability analysis for freeways or urban corridors.
- Production of service volumes.

When to use microsimulation. When an HCM limitation is encountered, or a more detailed method is needed, microsimulation can be utilized.

Important Notes: Units of key performance measures used in both the HCM and in microsimulation are not necessarily the same. Microsimulation will require adjustments if HCM thresholds are used for determining LOS. For example, the control delay derived from microsimulation should include all the data beyond the 15-min AP to be qualified as an "HCM Control Delay" and be used to determine an intersection LOS.

For uninterrupted flow methods, density computed by simulation tools usually refers to density in vehicles per distance unit, while the density used as a service measure for freeway and multilane highway segments refers to equivalent flow rate, measured in passenger car equivalent, including truck equivalencies; PHF; and other adjustment factors, as appropriate.

It is a good practice to compare results from simulation tools with example problems to ensure underlying parameters are modified to match those in the HCM. Also note that due to differences in assumptions, locally calibrated microsimulation model results may diverge from calibrated HCM-based model results. Refer to alternative tool information throughout the HCM and in "Chapter 6 – HCM and Alternative Analysis Tools."

CHAPTER 3. PLANNING AND PRELIMINARY ENGINEERING APPLICATION GUIDE (PPEAG)

PURPOSE OF THE PPEAG

The Planning and Preliminary Engineering Applications Guide (PPEAG) to the HCM is designed to improve planning practice by identifying appropriate techniques for using the HCM in planning and preliminary engineering analyses and to illustrate these techniques through case studies. It is intended to be used by planners, engineers, and system analysts at various stages of the system management, operation, planning, and project development process.

The PPEAG is a reference and educational resource on best practices for applying HCM methods to a variety of planning and preliminary engineering applications. The PPEAG is part of HCM Volume IV and is freely available from https://hcmvolume4.org/.

APPLICATION OF THE PPEAG

PPEAG defines how it is to be used as follows:

- When applying HCM and HCM-consistent methods to a broad spectrum of planning and preliminary engineering applications (including different stages of project planning and development, various study area sizes, under- and over-capacity conditions, and system performance monitoring).

- When seeking to understand the appropriate use of default values when applying HCM methods, along with techniques for developing and using local default values.

- When coordinating the use of the HCM with simulation models, travel demand forecasting models, mobile source emissions models, multimodal transportation analysis tools, and planning tools.

- When desiring to incorporate and test more factors in analysis than traditional planning tools allow by integrating HCM methods with existing tools.

- When needing to simplify calculations to produce a quicker, more transparent evaluation and review process without sacrificing the accuracy of the conclusions.

TARGET USERS OF PPEAG

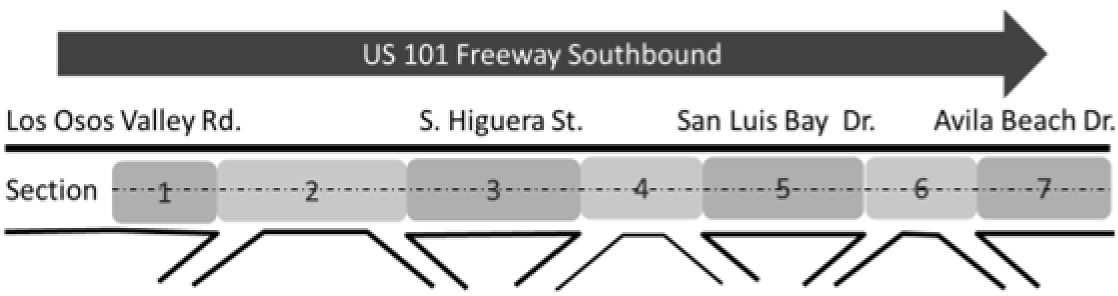
The range of potential users for the PPEAG includes every technical professional involved in estimating the need for, and feasibility of, highway capacity, monitoring, management, and operations investments. This audience includes all current HCM users, plus planners and travel demand modelers who may not consider themselves HCM users but who have used pieces of the HCM in the past. University students in transportation planning and transportation engineering programs are also part of the target audience.

EXAMPLE CASE OF HOW TO CONDUCT PPEAG ANALYSES

The purpose of this example is to demonstrate the PPEAG analysis. The main difference between PPEAG and HCM methods is that performing the HCM method for freeways requires software due to complexity. However, while the PPEAG method can be executed without software, the use of spreadsheets may be inevitable.

A stretch of freeway in California was selected to perform a PPEAG analysis. The owning agency's planning objective is to develop a corridor master plan to identify current and future mobility problems and establish capital project priorities along the corridor.

As shown in figure 24, the freeway is divided into several sections (**not HCM segments**), bounded around the ramps' gore points. The simpler section types (basic and ramps) provide a less complex spatial unit for the analysis. The necessary inputs for the analysis pertaining to geometry and traffic demand are also provided in the figure below.



Section	C-1	C-2	C-3	C-4	C-5	C-6	C-7
Section type	Basic	Ramps	Basic	Ramps	Basic	Ramps	Basic
Length (mi)	0.05	1.65	0.24	1.51	0.37	0.81	0.18
Lanes	2	2	2	2	2	2	2
Mainline AADT	41,700						
On-ramp AADT		8,600		6,100		1,400	
Off-ramp AADT		500		4,600		1,400	
***K*-factor**	0.08						
% heavy vehicles	6%						
Free-flow speed	65 mph						
PHF	0.92						

Source: PPEAG

Figure 2. Illustration. PPEAG example case.

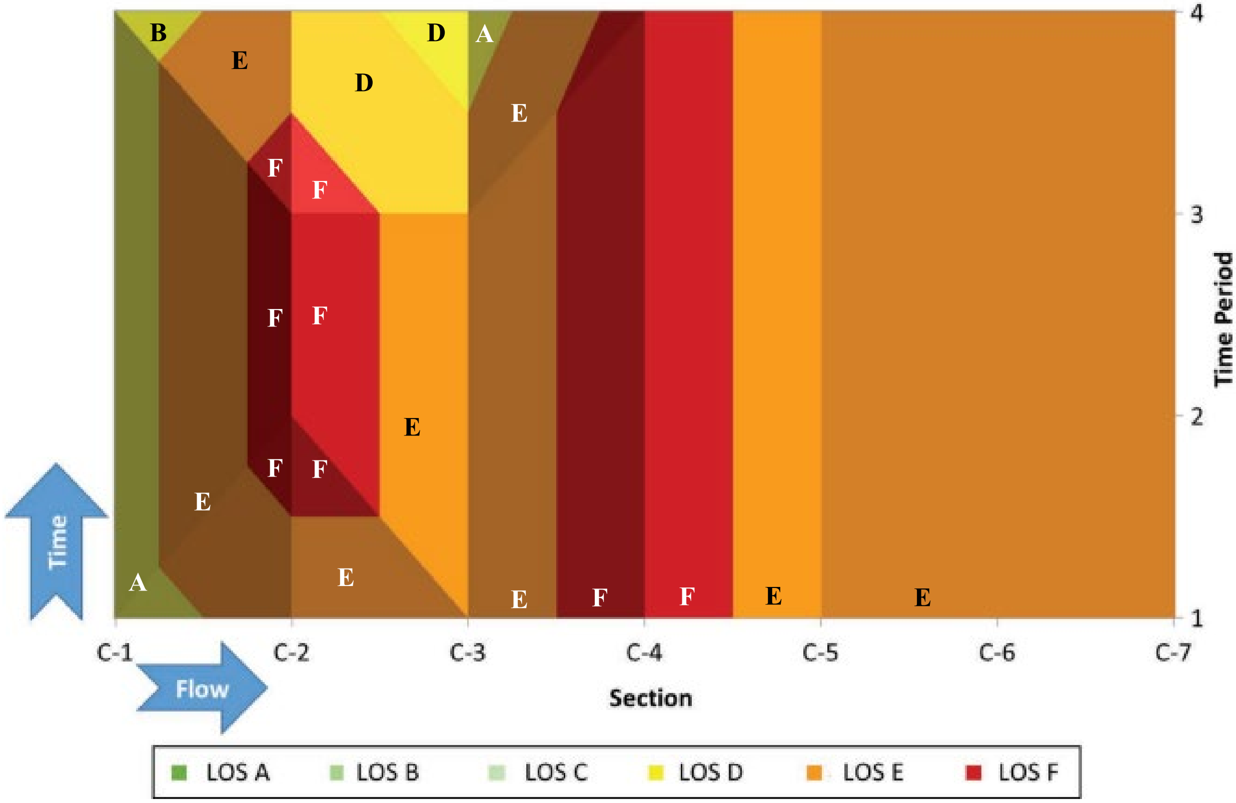
By performing PPEAG analysis methods on the basis of freeway sections and 15-min APs, the main MOEs can be estimated. Figure 35 shows the speed and travel time for the four consecutive 15 minutes analysis periods.

Section	C-1	C-2	C-3	C-4	C-5	C-6	C-7
Section type	Basic	Ramps	Basic	Ramps	Basic	Ramps	Basic
Length (mi)	0.05	1.65	0.24	1.51	0.37	0.81	0.18
Time Period 1							
Undersat. delay rate (s/mi)	1.7	10.2	6.9	13.5	5.6	9.5	5.6
Oversat. delay rate (s/mi)	0	0	0	18.4	0	0	0
Travel time (s)	2.9	108.3	14.9	131.7	22.6	52.6	11.0
Speed (mph)	62.1	54.8	58.0	41.3	58.9	55.4	58.9
Time Period 2							
Undersat. delay rate (s/mi)	3.5	13.5	9.3	13.5	5.7	9.8	5.7
Oversat. delay rate (s/mi)	0	10.4	0	53.0	0	0	0
Travel time (s)	2.9	130.9	15.5	184.0	22.6	52.8	11.0
Speed (mph)	62.1	45.4	55.7	29.5	58.9	55.2	58.9
Time Period 3							
Undersat. delay rate (s/mi)	1.7	13.0	8.9	13.5	6.2	10.4	6.2
Oversat. delay rate (s/mi)	0	0	0	82.8	0	0	0
Travel time (s)	2.9	112.8	15.4	229.1	22.8	53.3	11.1
Speed (mph)	62.1	52.7	56.1	23.7	58.4	54.7	58.4
Time Period 4							
Undersat. delay rate (s/mi)	0.6	5.6	3.6	13.5	6.4	10.6	6.4
Oversat. delay rate (s/mi)	0	0	0	73.7	0	0	0
Travel time (s)	2.8	100.7	14.1	215.3	22.9	53.4	11.1
Speed (mph)	64.3	59.0	61.3	25.2	58.2	54.6	58.4

Source: PPEAG

Figure 3. Chart. PPEAG example case results.

In addition to speed and travel times, other performance measures can be derived as well. As an example, figure 46 shows a heatmap of HCM LOS for this example case, where the measure is calculated based on density (flow divided by speed) for each section.



Source: PPEAG

Figure 4. Chart. Example case results – LOS heatmap.

CHAPTER 4. BASIC FREEWAY SEGMENTS – HCM CHAPTERS 12 AND 26

Chapter 12 is used to analyze one direction of travel at a time for a basic freeway segment. Additional analysis is necessary to model the opposing direction. Segments should be homogenous and broken into multiple analyses if noteworthy operating features (e.g., number of lanes, free-flow speed, clearances, grades, among others) vary significantly.

VARIABLES AND MODEL ASPECTS

FFS: The default value for FFS is 75.4 mi/h. For operational analysis, the FFS should ideally be field-measured. Field-measured free-flow speed becomes very important in locations where geometric design has lower standards for lateral clearance or grade (i.e., urban or mountainous situations) since it could be significantly lower than the 75.4 mi/h default (which can be overridden as of the Update). When FFS cannot be measured, or for planning and design analyses, the HCM method estimates FFS based on a base free-flow speed (BFFS) and the following parameters:

- *Lane Width*: The lateral distance between stripes for a given lane, measured in feet (Default 12 ft / Range 10–12 ft / Typical 12 ft).

- *Lateral Clearance*: While right-side lateral clearance provides an adjustment to FFS, left-side lateral clearance issues are ignored in the procedure and may represent another potential need to measure free-flow speed (Default 6 ft/ Range 0–10 ft/ Typical 6 ft).

- *Ramp Density*: Ramp density is determined by counting the ramps (not interchanges) 3 mi upstream and downstream from the analysis segment midpoint, then dividing by six to obtain the ramps per mile (Range 0–6).

- *Truck Population*: Besides the percentage of heavy vehicles relative to the total traffic, the heavy-vehicle mix is defined as the split between single-unit trucks (SUT) (FHWA classifications 4 and 5) and tractor-trailers (TT), with buses and recreational vehicles considered SUTs. The values for percentage of SUTs and percentage of TTs are entered as the proportion of each heavy vehicle type relative to the total truck population when analyzing specific grades. The terrain type (e.g., level, rolling, or specific grades) has a significant impact on truck performance as well. These inputs are used to calculate passenger car equivalents (PCE) (FHWA classifications 6–13).

However, the PCE factors may not be accurate when at least one of these conditions exists:

- Significant presence of trucks in the traffic stream.
- A long upgrade.
- A combination of both conditions.

The HCM supplemental chapter 26 presents the mixed-flow model to address this issue in freeway basic segments and multilane highways. While the PCE approach assumes vehicle speeds are uniform across all vehicle types, the mixed-flow model calculates speeds for passenger cars and trucks individually, yielding more accurate results.

For low percentages of trucks and mild upgrades, the results provided by the PCE methodology are comparable to those provided by the mixed-flow model. The HCM, however, does not provide a strict definition of the values at which the percentage of heavy vehicles and grade are considered significant to support a mixed-flow analysis. Hence the agency should have the discretion to require this method when adequate.

MLs: MLs are modeled for five types of designs, including:

- Continuous access (single lane, skip or solid stripe).
- Buffer 1 (buffer-separated, single lane).
- Buffer 2 (buffer-separated, multiple lanes).
- Barrier 1 (barrier-separated, single lane).
- Barrier 2 (separated, multiple lanes).

Capacity values for MLs within basic, weaving, merge, diverge and access segments are provided as functions of the flow speed and access design. When the density of the general-purpose lanes exceeds 35 passenger cars per mile per lane (pc/mi/ln), friction is assumed, and the speed-flow curves are adjusted for the continuous access and buffer single-lane buffer designs.

Adjustments: Capacity and speed adjustment factors (CAF and SAF) are provided for weather events and driver population mix, and CAF are also provided for incident events. The 7th Edition of HCM contains additional adjustment factors for CAVs (CAF_{CAV}). CAF_{CAV} ranges from 1.00 to 1.414 as a function of CAV market penetration.

Average Travel Speed (*S*): Basic speed–flow curves have been developed for FFS values between 55 and 75 mi/h for freeways. In the HCM, equation 12-1 and exhibit 12-6 values are used directly to calculate the average travel speed.

Traffic Density: Traffic density is the service measure used to define LOS for basic freeway segments. It is computed by the fundamental relationship between equivalent traffic flow and average speed.

Adjusted Capacity and LOS F: The segment capacity is calculated from the base capacity and adjusted by all the CAFs considered by the analyst. LOS F is assigned whenever the equivalent flow rate is greater than the adjusted capacity.

REVIEW CHECKLIST

Table 74 serves as a checklist that the analyst or reviewer should look for in the HCM analysis.

Table 7. Checklist for basic freeway segments.

Basic Freeway Segments Topics	Type	Default	✓
One direction	**Modeling**	-	
Free-flow speed	Input	Speed limit + 5 mi/h	
Lane width	Input	12 ft	
Lateral clearance	Input	10 ft	
Ramp density	Input	No default/User must provide	
Terrain type (level, rolling or grade)	Input	No default/User must provide	
Truck population and mix model	Input	5% (urban); 12% (rural)	
MLs	**Modeling**	-	
Weather adjustment factors	Calibration	1.0	
Incident adjustment factors	Calibration	1.0	
Driver population	Calibration	1.0	
CAV market penetration	Calibration	0%	
Average travel speed	MOE	-	
Traffic density	**Service measure**	-	
Capacity and LOS F	**MOE**	-	

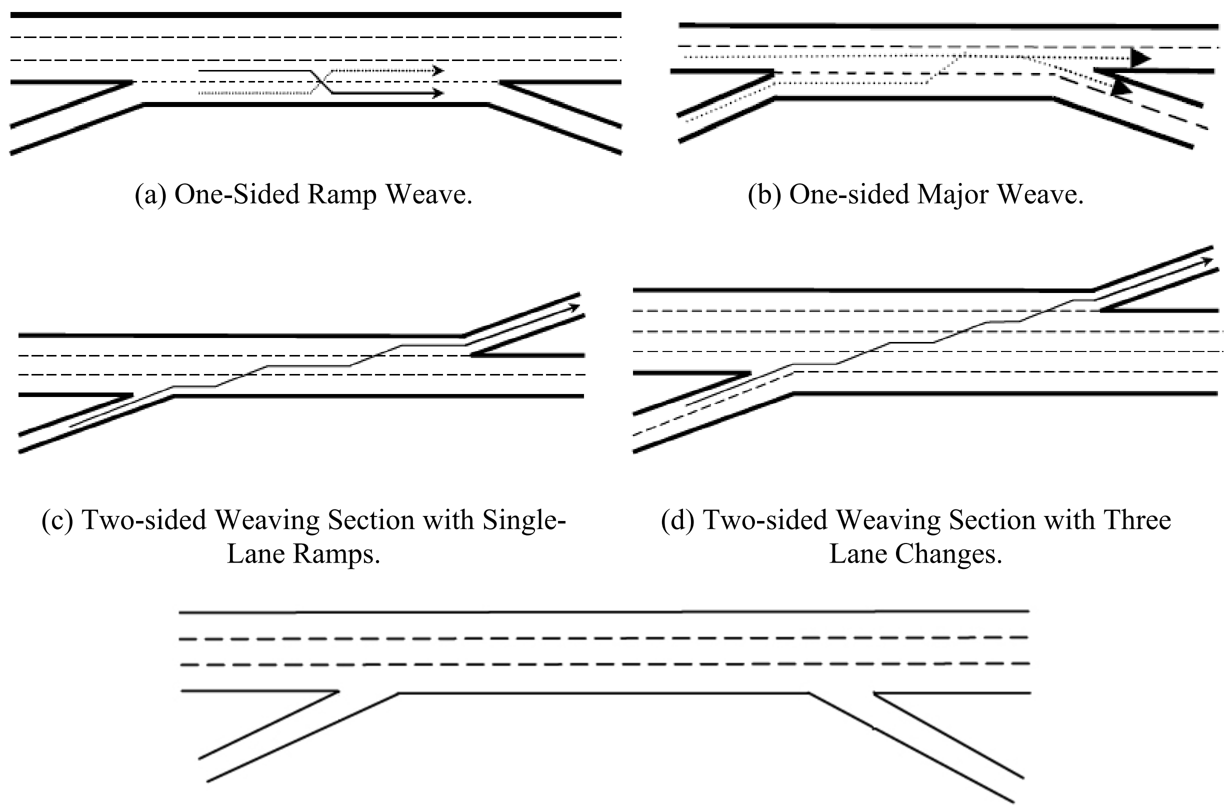
HCM LIMITATIONS

The HCM methodology for basic freeway segments does not apply to or take into account (without modification by the analyst) the following:

- Lane controls (to restrict lane changing).
- Extended bridge and tunnel segments.
- Segments near a toll plaza.
- Segments with an FFS of more than 75.4 mi/h.
- Segments with a base FFS less than 55 mi/h for freeways, although lower FFS values can be achieved by calibrating an SAF.
- Posted speed limit and enforcement practices.
- Presence of intelligent transportation systems (ITS) related to vehicle or driver guidance.
- Capacity-enhancing effects of ramp metering.
- The influence of downstream queuing on a segment.
- Operational effects of oversaturated conditions. Analysts should be directed to the facilities analysis for oversaturated conditions for all segment chapters.

CHAPTER 5. FREEWAY WEAVING SEGMENTS – CHAPTERS 13 AND 27

The HCM presents methods for the analysis of one-sided and two-sided weaves. One-sided weaves are defined by an on-ramp followed by an off-ramp connected by a continuous auxiliary lane. Two-sided weaving segments are formed by closely spaced on- and off-ramps on the opposite sides of the freeways, as shown in the figure. This configuration is the first aspect to be checked while reviewing a weaving analysis with HCM. When these conditions are not met, the merge and diverge areas operate independently. Overlap segments are segments between the area of influence of an on-ramp and an off-ramp, which do not form a conventional weaving.



(a) One-Sided Ramp Weave.

(b) One-sided Major Weave.

(c) Two-sided Weaving Section with Single-Lane Ramps.

(d) Two-sided Weaving Section with Three Lane Changes.

(e) Merge, Basic, Diverge (not Weaving without Auxiliary Lane).

Source: FHWA.

Figure 5. Illustrations. Weaving length measures and.

VARIABLES AND MODEL ASPECTS

Weaving-segment analysis shares the same basic assumptions with basic freeway segments, including unidirectional analysis, FFS speed measurement, and speed and capacity estimation principles and adjustment factors (SAF and CAF) based on driver population, weather, incidents, and CAV market penetration. Weaving-specific aspects are discussed in the following subitems. This procedure can be applied to multilane highways and collector-distributor roadways, with the only change being a more compressed LOS table.

Weaving Demand: In addition to freeway volume, a weaving analysis requires origin-destination (OD) demand between ramp entrances and exits. Therefore, demand must be coded for freeway-to-freeway, ramp-to-freeway, freeway-to-ramp, and ramp-to-ramp movements. All further calculations are highly sensitive to these values.

OD demand may be difficult to collect on the field. The user is advised to use caution when analyzing results with estimated demand data.

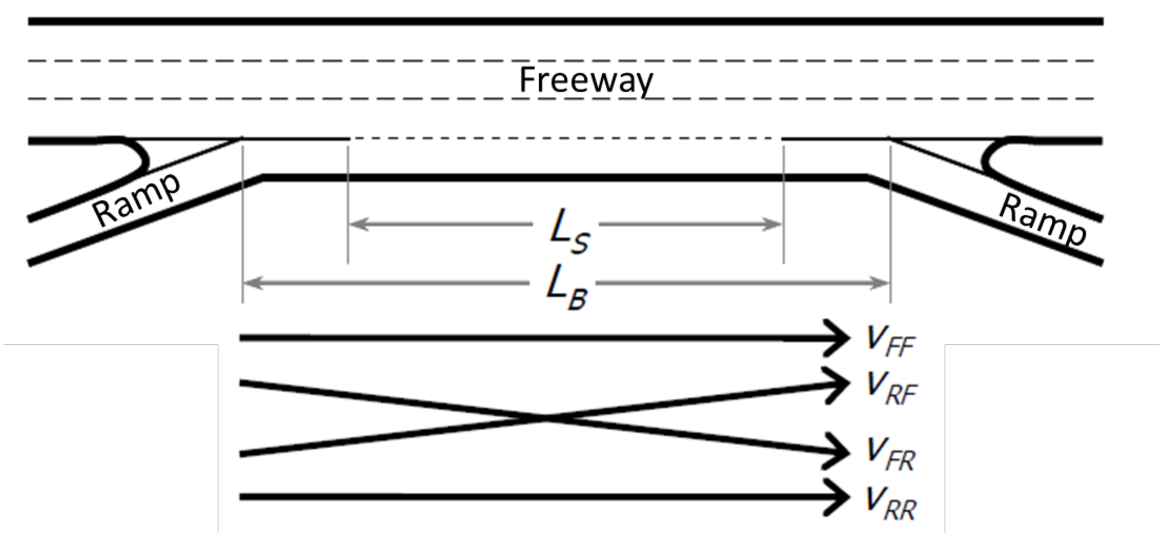
Weaving Lanes: The number of weaving lanes is defined based on the number of lane changes required for each movement. The auxiliary lane is included in the total number of lanes for one-sided weaves, while only the number of freeway lanes is used for two-sided weaves. The number of lanes that can make a maneuver to get on or off the freeway with zero or one-lane change is an important factor affecting the weaving analysis.

Weaving Length: There are two measures of weaving segment length:

- Short Length (L_S) = the distance in feet (or meters in metric) between the endpoints of any barrier markings (solid white lines) that prohibit or discourage lane changing.

- Base Length (L_B) = the distance in feet (or meters in metric) between points in the respective gore areas where the left edge of the ramp-traveled way and the right edge of the freeway-traveled way meet.

The total weaving segment length is the base length plus a 500-ft influence area downstream and a 500-ft influence area upstream of merge and diverge points.



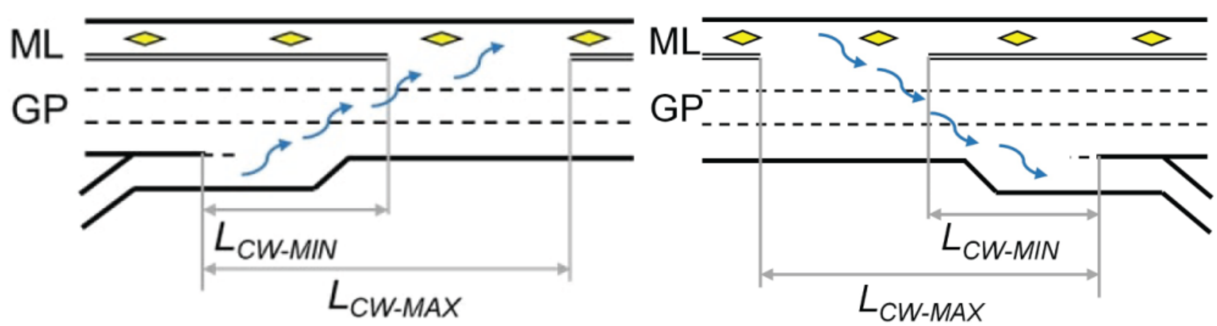
V_{FF} = freeway-to-freeway demand flow rate in the weaving segment. V_{FR} = freeway-to-ramp demand flow rate in the weaving segment. V_{RF} = ramp-to-freeway demand flow rate in the weaving segment. V_{RR} = ramp-to-ramp demand flow rate in the weaving segment.
Source: HCM exhibits 13-2 and 13-9.

Figure 6. Illustration. Weaving length measures and required demand.

Maximum Weaving Length: The HCM computes the maximum weaving length (L_{MAX}) as an intermediate variable. It reflects the maximum length where the segment would operate as a weaving segment. If the L_S is greater than L_{MAX}, then the segment should be analyzed using the basic freeway segment method in the HCM. (The HCM points out the limitation of this value to sometimes be too high and recommends some level of field or simulation confirmation that the friction between entering and exiting vehicles does exist.)

MLs and Cross-Weaving Segments: As an extension of the weaving methodology, the HCM presents methods to calculate the capacity reduction due to cross-weaving movements in freeway facilities. Cross-Weaving movements are defined as vehicle movements between an ML on the leftmost lane of the freeway and the nearby entry or exit ramps. This configuration causes the freeway general purpose lanes (GP) to operate as a weaving segment. The vehicle performance is also sensitive to road length (L_{CW}) availability to complete maneuvers.

The definition of the access point location for ML facilities becomes a design decision that affects operations along the facility. Research has shown an optimal access point may exist to maximize the performance of a cross-weave facility.[1] For example, where an off-ramp follows an on-ramp, the managed-lane access locations should be adjusted so that both the upstream merge and the downstream diverge have sufficient length to allow vehicles to move efficiently between ramps and access points. The cross-weave length is not an HCM output itself; rather, finding the optimum ramp locations is a trial-and-error process to balance these considerations and find the configuration that will yield the best performance.



L_{CW-MIN} = minimum cross-weave length. L_{CW-MAX} = maximum cross-weave length.
Source: FHWA.

Figure 7. Illustration. Cross-weaving configurations.

LOS: The thresholds defined for freeway weaving segments differ from those provided for basic freeway segments, with failure at 43 pc/mi/ln (40 pc/mi/ln for non-freeways) instead of 45 pc/mi/ln.

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[1] Dong, S., V. Khanapure, S. Taningco, and B. Sampson. (2017). "Optimize the Location of Managed Lanes Access Segment for Efficient Cross-Weaving in Freeway Facilities." Presented to the 5th Annual UTC Conference for the Southeastern Region, STRIDE, University of Florida, Gainesville, FL.

REVIEW CHECKLIST

Table 85 serves as a checklist that the analyst or reviewer should look for in the HCM analysis.

Table 8. Checklist for freeway weaving segments.

Freeway Weaving Segments Topics	Type	Default	✓
Weaving configuration (one or two-sided) length	Modeling	-	
Free-flow speed	Input	Speed limit + 5 mi/h	
Lane width	Input	12 ft	
Lateral clearance	Input	10 ft	
Ramp density	Input	No default/User must provide	
Terrain type (level, rolling or grade)	Input	No default/User must provide	
Truck population and mix model	Input	5% (urban); 12% (rural)	
Weaving lanes	Input	2	
Weaving demands	Input	No default/User must provide	
Weather adjustment factors	Calibration	1.0	
Incident adjustment factors	Calibration	1.0	
Driver population	Calibration	1.0	
CAV market penetration	Input	0%	
Maximum weaving length	Intermediate Output	Less than short length (L_s)	
MLs and cross weaving	Modeling	-	
Density	Service Measure	-	

HCM LIMITATIONS

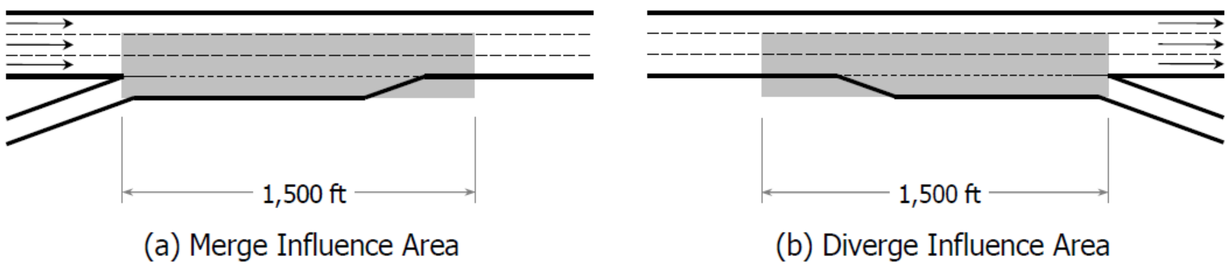
The HCM methodology for basic freeway segments does not apply to or take into account (without modification by the analyst) the following:

- Ramp metering on entrance ramps, which form part of the weaving segment.
- Segment speed and other performance measure estimation during oversaturated conditions for an isolated weaving analysis.
- Effects of speed limit enforcement practices on weaving segment operations.
- Effects of intelligent transportation system technologies on weaving segment operations.

- Overlapping weaving segments, which must be divided into the appropriate merge, diverge, and simple weaving segments for analysis.
- Weaving segments on urban streets and arterials, since urban streets are interrupted (not uninterrupted) flow analyses. Research is underway to develop a procedure for interrupted weaving analysis.

CHAPTER 6. FREEWAY MERGE AND DIVERGE SEGMENTS - HCM CHAPTERS 14 AND 28

In the practice of analyzing LOS, the terms “on-ramp” and “merge” are often used interchangeably, as are “off-ramp” and “diverge.” In the HCM, “Chapter 14 – Freeway Merge and Diverge Segments” can be used to analyze ramp junctions with the freeway mainline and merge or diverge points where two mainline roadways join or separate. The method is used to predict the operational conditions on merge and diverge influence areas, defined by the HCM as 1,500 ft from the merge/diverge points and only including the two freeway lanes adjacent to the ramp (figure 810).



Source: HCM exhibit 14-1.

Figure 8. Illustration. Merge and diverge influence areas.

VARIABLES AND MODEL ASPECTS

Merge and diverge segment analysis shares the same fundamental assumptions with the basic freeway segments, including unidirectional analysis, FFS speed measurement, and estimation principles and adjustment factors for speed and capacity (SAF and CAF) based on the driver, population, weather, incidents, and CAV market penetration. Merge- and diverge-specific aspects are discussed in the following subitems.

Merge and Diverge Lanes: Unlike weaving segments, the base number of lanes in a weaving merge and diverge segment is defined as the number of freeway lanes only. The acceleration and deceleration lane lengths must be entered as another variable, which will be used in different steps of the HCM method.

The number of on-ramp and off-ramp lanes is used for ramp capacity checks. If ramp capacity is less than demand on a diverge segment, LOS F is expected on the mainline, while congestion on an on-ramp will limit the entry demand on the freeway mainline.

Merge and Diverge Demand: In addition to freeway volume, a merge or diverge analysis requires the on-ramp (merge) or the off-ramp (diverge) demand values.

Left-Hand Ramps: When ramps are on the left side of the freeway, the number of vehicles in the lanes closest to the ramps is determined by computing the volume in Lanes 1 and 2 for right-hand ramps with a final adjustment. The logic behind this increase in the estimate is that more through traffic will remain in lanes closest to the ramp when it is on the left than would be the case when all parameters are the same for a right-hand ramp. This is because through traffic typically stays to the left to avoid ramp friction but will not normally move to the right to avoid left-hand ramp friction since it's such a rare occurrence.

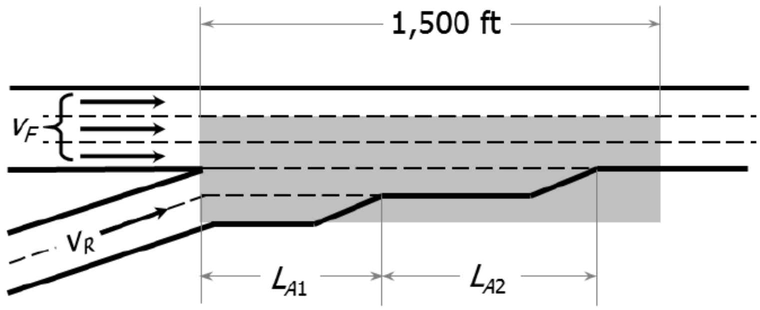
Adjacent Ramps: The effects of adjacent upstream and adjacent downstream ramps are only modeled for single-lane ramps on six-lane (three lanes in each direction) freeways. In these cases, the equilibrium distance (L_{EQ}) is used to determine which equation to use for computing the proportion of vehicles in Lanes 1 and 2 (P_{FM} or P_{FD}) immediately upstream of the ramp. Only one-lane right-side off-ramps can affect merges, and only one-lane upstream on-ramps and one-lane downstream off-ramps can affect diverges. If both adjacent upstream and downstream ramps exist, the analysis resulting in the highest proportion is used.

MLs: Left-side ramps directly interacting with an ML are analyzed as a special case under the ML methodology in the HCM 7th Edition (p. 14-35) using an adaptation of the methods in this chapter. This HCM adaptation accounts for the fact that there is no interaction between GP lanes and the MLs in the vicinity of the ramp. The operation of an ML merge or diverge segment with a single mainline lane can be approximated by doubling the ML mainline volume before analysis and evaluating the segment as if there were two through lanes on the MLs. The resulting computational results for segment speed and density will then be true to the assumptions used in the development of the methods in this chapter. The results should then be applied only to the single ML.

Density in the Influence Area: Density on the two rightmost lanes of the merge/diverge influence areas (two leftmost lanes or left-handed ramps) is the service measure for merge/diverge analysis. As acceleration and deceleration lengths increase, density decreases, which is expected. However, when the length of an acceleration or deceleration lane is greater than 1,500 ft, care should be taken to review density results. If these lengths get too long, density values can become unreasonably low, especially for two-lane ramps where the effective length can be longer than expected.

Aggregated Density: Aggregated density can also be computed across all lanes by dividing the total flow rate by the average speed in all lanes.

Lane Additions and Drops: One case covered by HCM deals with lane additions or drops for merges and diverges, respectively (p. 14-30). In these cases, the merge/diverge segment should be treated as a basic freeway segment with the appropriate number of lanes. Merges with two-lane ramps are not subject to the same rule per the HCM (pp. 14-30 to 14-31). The two-lane entrance or exit is characterized by two separate acceleration or deceleration lanes, each successively forcing merging maneuvers to the left, thus forcing the merge chapter to be used as long as the downstream freeway number of lanes is unchanged.



L_{A1} = auxiliary lane addition 1. L_{A2} = auxiliary lane addition 2. V_F = freeway volume. V_R = ramp volume.
Source: HCM exhibit 14-16.

Figure 9. Illustration. Two-lane ramp-freeway junction.

Major Merge and Diverge Areas: In a freeway analysis, diverge segments with an optional lane and no clear deceleration lanes may be considered major diverge areas. Major diverge areas are a special case covered by the HCM 7th Edition chapter 14 methodology, which will result in a modified density at the ramp influence area (p. 14-34).

Note that this method is a simplification, and simulation is advisable for complex major diverges and freeways with high demand values.

There is no HCM procedure for major merge configurations.

As with freeway basic segments, the merge and diverge methodology of the HCM 7th Edition provides capacity adjustment factors as a function of CAN market penetration.

REVIEW CHECKLIST

Table 96 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 9. Checklist for freeway merge & diverge segments.

Freeway Merge & Diverge Segments Topics	Type	Default	✓
Free-flow speed	Input	Speed limit + 5 mi/h	
Lane width	Input	12 ft	
Lateral clearance	Input	10 ft	
Ramp density	Input	No default/User must provide	
Terrain type (level, rolling or grade)	Input	No default/User must provide	
Truck population and mix model	Input	5% (urban); 12% (rural)	
Merge & diverge configuration	Modeling	No default/User must provide	

Freeway Merge & Diverge Segments Topics	Type	Default	✓
Freeway and acceleration/deceleration lanes	Input	No default/User must provide	
Left-hand ramps	Modeling	No default/User must provide	
Merge/diverge demands	Input	No default/User must provide	
Free-flow speed	Input	35 mi/h	
Adjacent ramps	Input	No default/User must provide	
MLs	Modeling	-	
Weather adjustment factors	Calibration	1.0	
Incident adjustment factors	Calibration	1.0	
Driver population	Calibration	1.0	
Cav market penetration	Input	0%	
Density on the merge/diverge influence area	Service Measure	-	
Aggregated density	MOE	-	
Lane additions and drop	Modeling	-	
Two-lane ramp junctions	Modeling	-	
Major merge/diverge		-	

HCM LIMITATIONS

The methodology in this chapter does not take into account, nor is it applicable to (without modification by the analyst), cases involving:

- Special lanes, such as high-occupancy vehicle (HOV) lanes, as ramp entry lanes.
- Ramp metering or intelligent transportation system features.
- Level of police enforcement.

CHAPTER 7. FREEWAY FACILITIES - HCM CHAPTERS 10, 11, AND 25

Freeway facilities are formed by a sequence of basic segments, weaving segments, and merge and diverge segments. The HCM recommends a facility start and end with basic segments to constrain the analysis of the points of interest and potential bottlenecks in space.

VARIABLES AND MODEL ASPECTS

All variables of importance for each segment type should be checked for the appropriate segments, as described in the previous sections of this document. Additional facility-specific aspects are described in the following items.

Facility Segmentation: The HCM provides guidance on how to properly segment a freeway facility, as follows (p. 10-7):

- A new segment should be started whenever demand volume changes.
- A new segment should be started whenever capacity changes, including when:
  - A full or auxiliary lane is added or dropped.
  - The terrain changes significantly.
  - The FFS is expected to change (i.e., changes to lane widths or lateral clearance);
- The influence area of a ramp is considered to be 1,500 ft, measured downstream from the gore point for on-ramps and upstream of the gore point for off-ramps. The end of a merge segment's ramp influence area often represents a transition to a basic freeway segment. Similarly, a basic segment transitions to a diverge segment at the beginning of the ramp influence area. The exact location of the gore points is clear when physical barriers are present. Analyst judgment and considerations on local behavior may be needed to pinpoint the exact location of painted gore points.
- Ramp segments, including the ramp influence area, are classified either as merge or diverge segments.
- When two adjacent merge and diverge segments are connected by an auxiliary lane, the entire segment is coded as a weaving segment. The weave influence area extends 500 ft upstream and 500 ft downstream of the two respective gore areas.
  - When the gore-to-gore length between two adjacent merge and diverge segments exceeds 3,000 ft and no auxiliary lane exists, the section should be coded as a series of three segments (merge, basic, diverge). The basic segment length is the difference between the gore-to-gore spacing and 3,000 ft.
  - When the gore-to-gore length of two adjacent merge and diverge segments is less than 3,000 ft but longer than 1,500 ft and no auxiliary lane exists, the section should be coded as a series of three segments, with the middle segment being

defined as an overlap segment (merge, overlap, diverge) and used with both the merge and diverge segments to satisfy the 1,500 ft influence areas. In this case, the overlap segment length is the difference between 3,000 ft and the gore-to-gore spacing, and the merge and diverge segment lengths are equal to the gore-to-gore spacing minus 1,500 ft.

- If the ramp spacing is less than 1,500 ft without the addition of an auxiliary lane to connect the two gore areas, the 1,500 ft merge or diverge segment length is truncated at the adjacent ramp gore point.

- Any remaining unassigned segments that have been defined after all merge, diverge, weave, and overlap segments are labeled as basic segments.

Analysis Length: The analysis must be limited to the length of a freeway in which a vehicle can travel at average speed within 15 min, usually 9–12 mi.

Facility Capacity and Calibration: Capacity values are generally obtained from the individual segment methodologies and may not be representative of the local area situation. Capacity should be measured locally at bottleneck locations to determine more appropriate values for a given jurisdiction. For each segment, adjustment factors accounting for the weather, incidents, driver population and proportion of CAVs may be used if needed.

Queue Discharge Capacity Drop. When a breakdown occurs at a bottleneck, the queue discharge occurs at a rate lower than the segment's capacity (a 7-percent drop, based on a national average). This drop in capacity is a required input when analyzing freeway facilities.

Oversaturated Conditions: Modeling multiple segments over multiple time periods (with time periods of 15 minutes recommended) is required to model oversaturated conditions.

The analysis must begin and end as undersaturated, with the first and last segments not operating at LOS F, and the first and last AP may contain no segments operating at LOS F. For example, if the first segment is operating at LOS F and has a queue extending upstream and if the last segment is operating at LOS F with a downstream bottleneck, these segments should be analyzed as part of the facility, ideally including an upstream uncongested segment. Similarly, if the first AP is at LOS F, previous time periods could be failing. Conversely, if the last AP is at LOS F, subsequent periods may be failing. (HCM pp. 10-17, 10-34, and 10-35)

Work Zones: Work zones are modeled to generate CAF and *SAF* values to account for the effects of work zones. Parameters considered in developing these factors include a lane closure severity index comparing normal and open lanes; closure type (lane or shoulder); barrier type (concrete or drums); area type (urban or rural); lateral distance (travel lane to barrier); speed ratio (normal to work zone); time (day or night); and total ramp density. (HCM exhibit 10-15 and equations 10-8 through 10-12)

Travel Time Reliability: This methodology provides for the generation of a statistical distribution of trip travel time over an extended period as affected by variations in demand, weather, work zones, incidents, and special events on a freeway facility.

- Base Data Set – Segments and periods are defined in a complete freeway facility analysis as the basis for the generation of scenarios.

- Demand – Distribution of values by time of the day, day of the month, and month of the year.

- Weather – Nearest city for the provided database is selected for the most appropriate distribution of weather events by month for precipitation, snowfall, and temperature variations.

- Incidents – Types, locations, and severity proportions are provided in terms of frequency, response times, and clearance times.

- Special Events – Specific times and effects on demand are defined.

- Work Zones – Specific project locations, times, durations, and work zone modifications are defined.

- Scenario Generation – Based on the desired number of periods, unique combinations of demand, capacity, geometry, and traffic control conditions are produced to provide the distribution of results from which to compute the analysis parameters for describing travel time reliability.

- TTI – TTI is defined as the ratio of the actual travel time on a facility to the travel time at the base free-flow speed.

- Planning Time Index (PTI) – PTI is defined as the ratio of the 95th percentile highest travel time to the travel time at the base FFS. (HCM chapter 11)

ATDM: ATDM tactics can be evaluated adapting the facility configuration and controls to react to variations in demand, weather, and incidents. These might include changes to speed control, modifications to geometric configurations, etc. (HCM chapters 11 and 25)

MLs: Implemented for each segment within the facility as defined by the segment procedures. (HCM pp. 10-46 through 10-47)

Truck Procedure: Implemented for each segment within the facility as defined by the segment procedures. (HCM pp. 10-21 and 10-24)

Scale Factor: Flow balancing is achieved by comparing entering and exiting demand to generate the time interval scale factor that should approach 1.0. The scale factor is used to adjust the demand for each segment to balance any discrepancies. (HCM pp. 10-28 and 10-29 and equations 10-2 and 10-3)

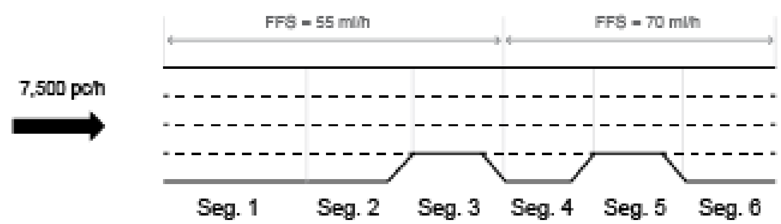
Facility Capacity: The capacity of the critical segment defines the capacity of the freeway facility. The critical segment is the one that will break down first. This means the first segment where demand exceeds capacity, not necessarily the segment with the lowest capacity.

The definition of the critical segment is within the analysis of the entire facility and depends on relative demands that can change among time periods.

Bottlenecks: The effect of a bottleneck (queuing affects upstream segments and capacity restraints meter downstream segments) on adjacent segments and the facility is extremely important in understanding the results when queuing and delay are part of the freeway system. However, an important question follows: will treating a bottleneck actually reduce congestion? To answer this question, it is crucial to understand the concepts of "Active bottleneck" and "Hidden bottleneck" and how to identify them correctly.

An active bottleneck has a demand greater than capacity, but the actual volume served in this location is metered by its capacity. Due to this metering effect of active bottlenecks, downstream segments will experience a demand shorter than the real demand. One of these downstream segments might have a capacity smaller than the actual demand but still greater than the volume served by the active upstream bottleneck. This is the concept of a "hidden bottleneck," which is not easily identifiable by field measurements but can be identified using HCM methods.

A short example below illustrates how can hidden bottlenecks be identified. Figure 10 shows a short freeway facility with six basic segments and an entering demand of 7,500 pc/h. Since there are no ramps in this facility, the demand for all segments is also equal to 7,500 pc/h. Segments 3 and 5 have a lane drop, making them potential bottleneck candidates. The measured free-flow speed is 55mi/h for segments 1–3 and 70 mi/h for segments 4–6.



	Segment 1	2	3	4	5	6
Capacity c (pc/h)	9,000	9,000	6,750	9,600	7,200	9,600
Demand flow rate v_d (pc/h)	7,500	7,500	7,500	7,500	7,500	7,500
Volume served v_a (pc/h)	7,500	6,285	6,285	6,285	6,285	6,285
v_d/c	0.83	0.83	**1.11**	0.78	**1.04**	0.78
v_a/c	0.83	0.70	**0.93**	0.65	0.87	0.65
LOS	D	**F**	**F**	C	**F**	C
Density (pc/mi/ln)	34.2	54.4	39.6	23.0	34.5	23.0
Speed (mi/h)	54.9	28.9	52.9	68.4	60.7	68.4

Source: FHWA.

Figure 10. Illustration. Active and hidden bottleneck analysis example.

Segment 3 is the first bottleneck met by a traveler in this facility. Although the segment capacity is 6,750 pc/h, this segment serves only 6,285 pc/h. This occurs due to the queue discharge capacity drop of 7 percent and can be confirmed by the ratio of volume served and capacity (va/c) of 0.93. Therefore, Segment 3 is an "active bottleneck," causing queues to extend upstream to Segment 2, which also yields an LOS of F.

Downstream of Segment 3, the arriving flow in subsequent segments is 6,285 pc/h because of the constraining bottleneck. Segment 5 has a capacity of 7,200 pc/h, resulting in a va/c of 0.87. Even though the volume served is below capacity, this segment still yields an LOS of F because the demand (7,500 pc/h) exceeds capacity (demand-to-capacity ratio = 1.04). Also note that the estimated speed in this segment is 60.7 mi/h, higher than Segment 3. Therefore, this segment is a "hidden bottleneck." It does not experience breakdowns due to the active upstream bottleneck, but as soon as the bottleneck in Segment 3 is removed, it becomes the next active bottleneck in the facility. Therefore, the congestion issue would not be solved, but would simply move to a new location.

In summary, for truly removing congestion, all active and hidden bottlenecks may need to be addressed. The analytical tools provided in the HCM and implemented in the Highway Capacity Software are a powerful tool for this purpose.

LOS: Results are interpreted from a matrix of values for multiple segments and APs to determine the worst situation for an overall LOS.

Note: LOS F can exist for a given segment when the queue from a downstream breakdown extends to that segment. LOS F can also be assigned to downstream hidden bottlenecks where the d/c ratio exceeds 1.0.

REVIEW CHECKLIST

Table 10107 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 10. Checklist for freeway facilities.

Freeway Facilities Topics	Type	Default	✓
Freeway segmentation	Modeling	-	
Area type (urban/rural)	Input	No default/User must provide	
Queue discharge capacity drop	Input	7%	
Jam density	Input	190 pc/mi/ln	
Weaving segments	Input	-	
Overlapping segments	Input	-	
Analysis length	Modeling	-	
Segment capacity adjustment and calibration	Calibration	-	
Critical/facility capacity	Intermediate output	-	
Oversaturated conditions	Modeling	-	
Work zones	Modeling	-	

Freeway Facilities Topics	**Type**	**Default**	✓
Active transportation demand management	Modeling	-	
MLs	Modeling	-	
Truck procedure	Modeling	-	
Travel time reliability	Modeling	-	
Active and hidden bottlenecks	Output	-	
Segment densities	**Service measures**	-	
Segment and facility LOS	Output	-	
Work zone analysis	Modeling	-	
Truck population and mix model	Input	-	
Active and hidden bottlenecks		-	
Limitations		-	
LOS		-	

HCM LIMITATIONS

Discussion of the facility methodology in the HCM is limited in that it does not fully address the following issues: multiple overlapping bottlenecks, HOV lanes, toll plazas, or off-ramps queuing onto the freeway.

In the HCM 7th Edition, “Chapter 38 – Network Analysis” does present new methods to address the interaction between freeway ramps and arterial corridors.

CHAPTER 8. MULTILANE HIGHWAYS – HCM CHAPTERS 12 AND 26

Chapter 12 is used to analyze one direction of travel on a multilane highway at a time. Additional analysis is necessary to model the opposing direction. Highways should be homogenous and broken into multiple analyses if noteworthy operating features (number of lanes, free-flow speed, clearances, grades, etc.) vary significantly.

VARIABLES AND MODEL ASPECTS

FFS: FFS should ideally be field-measured for operational analysis. When FFS cannot be measured, or for planning and design analysis, the HCM method estimates the FFS based on BFFS and the following parameters:

- *Lane Width*: The lateral distance between stripes for a given lane; measured in feet.
- *Lateral Clearance*: The sum of the left- and right-side lateral clearance provides an adjustment to FFS in conjunction with the number of lanes.
- *Median Type*: undivided, two-way left-turn lane (TWLTL) or divided.
- *Access Point Density*: The number of access points per mile is determined by dividing the total number of access points (i.e., driveways and unsignalized intersections) on the right side of the highway in the direction of travel by the length of the segment in miles. An intersection or driveway should only be included in the count if it influences traffic flow. Access points unnoticed by drivers or with little activity should not be used to determine access point density.

BFFS for multilane highways may be estimated, if necessary, as the posted or statutory speed limit plus 5 mi/h when speed limits are 50 mi/h and higher and as the posted or statutory speed limit plus 7 mi/h when speed limits are less than 50 mi/h.

Truck Population: Besides the percentage of heavy vehicles relative to the total traffic, the heavy-vehicle mix is defined as the split between SUT (FHWA classifications 4–5) and TT, with buses and recreational vehicles considered SUTs. The percentage of SUTs and percentage of TTs are entered as the proportion of each heavy vehicle type relative to the total truck population when analyzing specific grades. The terrain type (level, rolling, or specific grades has a significant impact on truck performance as well. These inputs are used to calculate the PCE (FHWA classifications 6–13)).

However, the PCE factors may not be accurate when at least one of these conditions exists:

- Significant presence of trucks in the traffic stream.
- A long upgrade.
- A combination of both factors above.

The HCM supplemental chapter 26 presents the mixed-flow model to address this issue in basic freeway segments and multilane highways. While the PCE approach assumes vehicle speeds are uniform across all vehicle types, the mixed-flow model calculates speeds for passenger cars and trucks individually, yielding more accurate results.

For low percentages of trucks and mild upgrades, the results provided by the PCE methodology are comparable to the ones provided by the mixed-flow model. The HCM, however, does not provide a strict definition of what values for percentage of heavy vehicles and grade are considered significant to support a mixed-flow analysis; hence, the agency should have the discretion to require this method when adequate.

Adjustments: CAF and SAF are provided for driver population mix.

Average Travel Speed (*S*): Basic speed–flow curves have been developed for FFS values between 45 and 70 mi/h for multilane highways. HCM equation 12-1 and exhibit 12-6 values are used specifically to calculate average travel speed.

Traffic Density: Traffic density is the service measure used to define LOS for basic freeway segments. It is computed by the fundamental relationship between equivalent traffic flow and average speed.

Adjusted Capacity and LOS F: The section capacity is calculated from the base capacity and adjusted by all the CAFs considered by the analyst. LOS F is assigned whenever the equivalent flow rate exceeds the adjusted capacity or the density exceeds 45 pc/mi/ln.

REVIEW CHECKLIST

Table 118 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 11. Checklist for multilane highways.

Multilane Highway Topics	Type	Default	✓
One Direction	**Modeling**	-	
FFS	Input	BFFS: Speed limit + 5 mi/h (50–70 mi/h) Speed limit + 7 mi/h (<50 mi/h)	
Lane width	Input	12 ft	
Right-side lateral clearance	Input	6 ft	
Median lateral clearance	Input	6 ft	
Access point density	Input	8 access points/mi (rural) 16 access points/mi (low-density suburban) 25 access points/mi (high-density suburban)	
Terrain type (level, rolling or grade)	Input	No default/User must provide	

Multilane Highway Topics	Type	Default	✓
Truck population and mixed model	Input	5% (urban) 12% (rural)	
Driver population	Calibration	1.0	
Average travel speed	MOE	-	
Traffic density	MOE	-	
Capacity and LOS F	**MOE**	-	

HCM LIMITATIONS

The HCM methodology for multilane highways does not apply to or take into account (without modification by the analyst) the following:

- Lane controls (to restrict lane changing).
- Extended bridge and tunnel segments.
- Segments near a toll plaza.
- Highways with an FFS of more than 70 mi/h or less than 45 mi/h.
- Posted speed limit and enforcement practices.
- Presence of ITS related to vehicle or driver guidance.
- Operational effects of oversaturated conditions.
- Operational effects of construction operations.

CHAPTER 9. TWO-LANE HIGHWAYS - HCM CHAPTERS 15 AND 26

Two-lane highways have one lane for the use of traffic in each direction. One direction is analyzed at a time, although the forward direction and opposing demand need to be coded for each direction. The single lane in each direction may be supplemented with passing lanes, truck climbing lanes, turnouts, or pullouts.

VARIABLES AND MODEL ASPECTS

The HCM 7th Edition includes a new methodological framework to analyze the quality of service of two-lane highways, based on the research project NCHRP 17-65 (Improved Analysis of Two-Lane Highway Capacity and Operational Performance).[2] The major highlights of the new methodology are described in the following items.

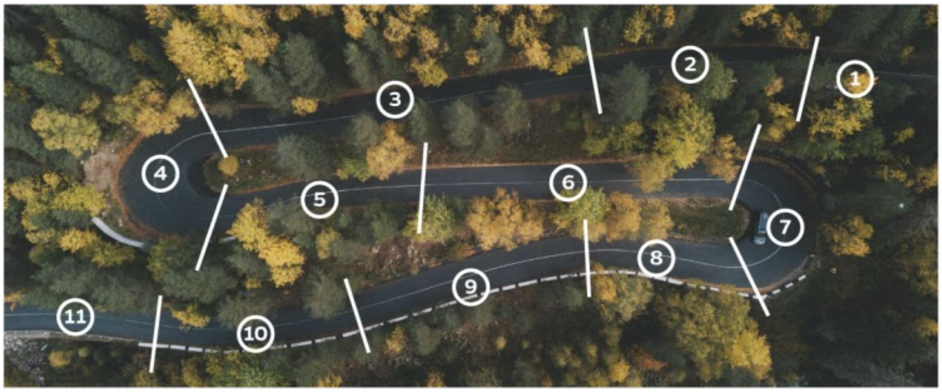
New Service Measure – Follower Density: The 2022 methodology introduced a new service measure called follower density (FD). It is described as "the number of vehicles in a follower state per mile per lane." In this context, a "follower" vehicle is defined by a headway equal to or less than 2.5 s. This service measure is used for any two-lane highway facility; no divisions in classes is necessary as in previous HCM editions. FD can also be measured directly on the field for calibration purposes.

Other performance measures included in the methodology include the following:

- Average Speed: average spot speed at the endpoint of the segment.
- Percent Followers: count of follower vehicles divided by the total number of vehicles.

Sensitivity to Horizontal Curvature: The 2022 methodology can now address horizontal curvature elements for more accurate speed estimations. A segment can be divided into sub-segments, classified as tangent or curve – for the latter, different horizontal curve classification groups are established based on curve radius (ft) and superelevation (percentage). Figure 11 shows an example of the definition of curve and tangent subsegments.

[2] Washburn, Scott S., Donald Watson, Zilin Bian, Ahmed Al-Kaisy, Amirhossein Jafari, Tapio Luttinen, Richard Dowling, and Aaron Elias. 2018. *NCHRP Web-Only Document 255: Improved Analysis of Two-Lane Highway Capacity and Operational Performance*. Washington, DC: Transportation Research Board of the National Academies. https://nap.nationalacademies.org/download/25179, last accessed November 1, 2022.



Subsegment	Type	Length (ft)[a]	Super-elevation (%)	Radius (ft)	Central Angle (deg)	Horizontal Class[b]
1	Tangent	280	--	--	--	--
2	Horizontal curve	432	3	450	55	3
3	Tangent	260	--	--	--	--
4	Horizontal curve	366.5	2	300	70	4
5	Tangent	250	--	--	--	--
6	Horizontal curve	216	5	275	45	5
7	Tangent	275.6	--	--	--	--
8	Horizontal curve	458	0	750	35	2
9	Tangent	285	--	--	--	--
10	Horizontal curve	767.9	4	1,100	40	1
11	Tangent	369	--	--	--	--
Total		3,960				

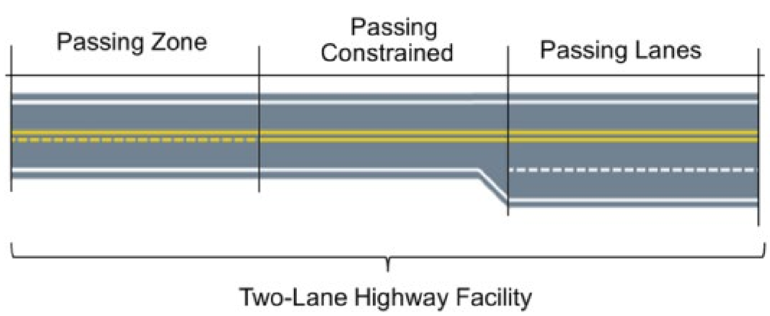
Source: HCM Exhibit 26-23.

Figure 11. Photo. Definition of curve subsegments.

Facility Analysis and Segmentation: The 2022 methodology allows the analysis of three different segment types, which can be aggregated into a facility-level analysis:

- Passing Zone: passing is permitted.
- Passing Constrained: passing is prohibited.
- Passing Lane: a second lane is added to a given direction of travel, allowing vehicles to pass without going into the opposing direction lane.

The percentage of no-passing zones and the length of any passing lanes within the segments are no longer used as inputs to the method as in previous versions of the HCM.



Source: FHWA.

Figure 12. Illustration. Two-lane highways segmentation framework.

FFS Estimation Based on Posted Speed Limit: The 2022 methodology redesigned the equations to estimate free-flow speed using the posted speed limit as an input when field measurements are not available. This approach is consistent with other Uninterrupted Flow methods in the HCM (freeways and multilane highways).

Analysis of 2+1 (Super 2) Sections: A Super 2 Highway, also called a "2+1 Highway," is a two-lane highway configuration with a continuous three-lane cross-section, with the middle lane being a passing lane that alternates direction. The new 2022 methodology also includes equations to measure the performance of this highway type.

REVIEW CHECKLIST

Table 12Table 12 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 12. Checklist for two-lane highways.

Two-Lane Highways Topics	Type	Default	✓
One direction	**Modeling**	**-**	
Segmentation and facility analysis	Modeling	-	
Passing lanes	Modeling	-	
Directional and opposing demand	Input	No default/User must provide	
FFS	Input	BFFS: 1.14 x speed limit	
Posted speed	Input	No default/User must provide	
Lane and shoulder width	Input	12 ft (lane width) 6 ft (shoulder width)	
Access point density	Input	0	
Curve and tangent subsegments	Input	No default/User must provide	
Heavy vehicle percentage	Input	6%	
Super 2 (2+1) design	Input	No default/User must provide	
Follower density	**Service Measure**	-	
Average speed	MOE	-	

HCM LIMITATIONS

The method does not consider the impacts of upgrades that begin before the analysis segment and continue through it, nor does it consider the additive impacts of multiple passing lanes. A microsimulation analysis is recommended in such situations to better capture the complex system effects.

The facility analysis methodology does not address two-lane highways with signalized intersections or other types of intersections requiring traffic on the highway to stop or yield. Isolated intersections on two-lane highways may be evaluated with the intersection methodologies. Two-lane highways in urban and suburban areas with multiple signalized intersections spaced 2 mi apart or less should be analyzed as urban streets using chapter 17.

CHAPTER 10. SIGNALIZED INTERSECTIONS – HCM CHAPTERS 19 AND 31

VARIABLES AND MODEL ASPECTS

Signal Operations: Traffic signals can be operated in fully actuated, coordinated-actuated, semi-actuated, coordinated-actuated, or pre-timed modes. Isolated signals usually operate in fully actuated or semi-actuated mode, while signals in close proximity along an urban street normally operate in coordinated-actuated or pre-timed mode.

- Fully Actuated – All phases operate with vehicle detection. A constant cycle length is used and phase durations for each movement vary depending on the volume under pre-defined maximum splits.
- Semi-Actuated – Detectors are used only for some approaches, usually the minor street movements, to serve traffic volumes accordingly. The major movements operate with a fixed duration. The signal continues to serve the major road when actuations are not received from the minor street approaches.
- Coordinated-Actuated – Only minor movements have detection with major movements operating as non-actuated for a duration calculated as the sum of the actuated phase times subtracted from the cycle length. Cycle length is kept constant to facilitate coordination, and each phase split varies with demand.
- Pre-timed – This control type uses a fixed sequence of phases that are displayed in a repetitive order. The duration of each phase is fixed. However, the green interval duration can be changed to accommodate traffic variations. The combination of a fixed phase sequence and fixed duration produces a constant cycle length. All phases have a fixed duration with no detection used. Green times are calculated between preset minimum and maximum values for better performance, and the cycle length becomes an output resulting from the sum of green, yellow, and red times for all phases.

Phasing must be modeled as it functions, and care must be taken to ensure the appropriate operation mode is used. For example, modeling a semi-actuated (uncoordinated with phases 2 and 6 in max recall) signal would generate different results than modeling a coordinated-actuated (coordinated for variable phases 2 and 6 with a fixed cycle length) signal (HCM pp. 31-1 and 31-2).

Lane Groups: Lane configuration needs to be entered for each approach. Approach lanes can be assigned for exclusive movements, or shared lanes may be used. Becoming familiar with all lane group possibilities as implemented can be important to understanding adjusted flow rates from the shared-lane model. The prediction of lane choice is based on drivers attempting to minimize service time, which creates an equilibrium that can be estimated from the lane-volume distribution that yields the minimum service time. Results for capacity, queue, delay, and level of service are reported by lane group initially, then weighted averages are used to generate results by approach and intersection (HCM exhibit 19-19).

Arrival Demand: Counting vehicles as they cross the stop line is inadequate for collecting data to analyze congested conditions. If demand approaches or exceeds capacity, arrival rate must be known to use demand in this methodology by collecting arrival data upstream of all queues associated with the approach, then reconciling the approach rate to each movement at the stop line.

Note: Another method is to quantify unmet demand at the beginning of the red phase for each movement for each cycle to determine the actual demand in oversaturated conditions. Unmet demand at the end of each period is added to the stop-line count after deducting that unmet demand from the previous period. The process for computing arrival demand from stop-line counts and unmet-demand queues is illustrated in Table 13.

Table 13. Quantification of unmet demand to estimate arrival demand.

Period	Stop-Line Count	Unmet Demand	Arrival Demand
1	400	0	400 = 400+0
2	500	50	550 = 500−0+50
3	500	75	525 = 500−50+75
4	400	0	325 = 400−75+0

If actual-demand data are not collected for congested conditions, the rate cannot exceed capacity (by definition) and the analysis can significantly underestimate delay and queue. For example, modeling any oversaturated movements using stop-line counts will not produce accurate results and should not be accepted; actual unmet-demand data should be required to verify that arrival rate (not departure flow) was measured.

Field data collection should include more than just turning-movement demand to adequately model signalized intersections. For the reasons described below, information on heavy vehicles, right turns on red (RTOR), parking maneuvers, bus stopping, lane utilization, pedestrians, and bicycles must be collected at the same time as the traffic counts are made (HCM p. 19-15).

RTOR: The value for RTOR must be obtained on the field or estimated. This value is subtracted from right-turn demand before the adjusted flow rate is computed.

Multiple-Period Analysis: If the signalized intersection is congested, a multiple-period analysis is required to properly model the operation for reasonable delay, queue, and LOS results. Otherwise, these computations do not consider the initial queue delay that builds and dissipates over the peak period. This analysis must begin and end with undersaturated periods to capture the complete oversaturation process.

Single versus multiple-period analysis comparisons illustrate the effects of unmet demand on delay and queue results that can differ by orders of magnitude. For example, table 14Table 14 illustrates that, for a v/c ratio of 1.5, the delay for the appropriate multiple-period analysis can be 372 percent of the inappropriate single-period analysis, whereas the queue for the appropriate multiple-period analysis can be 233 percent of the inappropriate single-period analysis.

Accepting these results would severely underestimate the costs to mitigate this congestion and could cause turn lanes to be under-designed (HCM pp. 19-19, 19-55, 19-57, and equation 19-44).

Table 14. Comparison between single-period and multiple-period analysis MOEs.

Delay (s/veh)				Queue (veh)			
v/c Ratio	Single Period	Maximum Multiple Period	Difference (percent)	v/c Ratio	Single Period	Maximum Multiple Period	Difference (percent)
0.24	35.10	35.10	0	0.24	5.20	5.20	0
0.42	38.30	38.30	0	0.42	8.70	8.70	0
0.56	41.70	41.80	0	0.61	12.60	12.60	0
0.79	38.30	38.30	0	0.79	17.20	17.20	0
0.96	77.40	78.60	2	0.97	24.50	24.50	0
1.16	138.60	284.30	105	1.16	63.10	101.00	60
1.35	198.70	452.40	128	1.35	38.40	86.40	125
1.54	302.60	1,427.00	372	1.54	49.90	166.00	233
1.74	391.20	1,287.00	229	1.75	63.20	270.00	327
1.87	448.20	2,294.00	412	1.91	73.80	342.00	363
1.99	498.9	2,452.00	391	2.07	84.70	365.00	331

s/veh = seconds per vehicle.

Unsignalized Movements: Delay of unsignalized movements is not computed by the HCM method but should be included as an input for computing approach and intersection aggregate delay and LOS. The delay for free-flow right turns is usually equal to zero and can be easily included in the analysis. Delay for other unsignalized movements needs to be estimated by means external to the HCM, such as direct field measurement, observation of similar conditions, special application of other models from the HCM, and simulation.

Control Delay: As the service measure for signalized intersections, control delay quantifies the increase in travel time due to traffic signal control by adding the delay incurred when decelerating to stop and accelerating back to traveling speed to the time stopped. It is also a surrogate measure of driver discomfort and fuel consumption.

Approach and Intersection Delay: Volume-weighted averaging among lane groups for approach delay, and among approaches for intersection delay, can generate misleading delay and LOS results. For example, two approaches with LOS A and two approaches with LOS F could produce an intersection LOS of C—but that would not be representative of the operation. In addition, adding traffic (as in a traffic impact analysis) to approaches with the least delay (previously undeveloped) could result in a reduction in the calculated intersection delay, which is also misleading.

Delay reported in s/veh do not account for the number of delayed vehicles. In other words, an average delay of 60 s/veh for a movement with a demand of 1,000 veh/h is considered equal to that same average delay of 60 s/veh when applied to a movement with a demand of 10 veh/h – but the effect on traffic is not the same at all. Computing vehicle hours of delay (average delay times demand divided by 3,600) can make for much better comparisons, especially when prioritizing improvement projects with an eye to the overall benefit to the public.

The HCM base procedures have limitations on modeling the effects of queue spillover on capacity. For turn lanes, the turn queue exceeding the storage will inhibit the adjacent through lane capacity, but this is not considered in the HCM computations of capacity, delay, and LOS in chapters 16 through 20. An analysis with the base methodology may provide only a queue storage ratio greater than 1.0 and lane blockage. The reviewer should assess whether these performance measures are sufficient for the project's purpose.

HCM supplemental chapters 29 and 30 (urban street facilities and segments, respectively) present a method to address sustained spillbacks and their effect on capacity and LOS. The method is more complex, and simulation may be recommended as an alternative to model this situation.

Base Saturation Flow Rate Calibration: Default values of 1,900 veh/hg/ln and 1,750 veh/hg/ln are provided for populations of over and under 250,000, respectively. (Default 1,900 / Typical 1,750 / Range 1,300–2,300).

The process for developing this parameter from field data is detailed in HCM chapter 31 and involves measuring the prevailing saturation flow rate for at least 15 cycles, including a minimum of 8 vehicles in-queue per cycle, excluding the first 5 vehicles (to account for start-up lost time) and permitted left-turn lane groups (because of the complexity involved).

This rate is compared with the computed rate to generate a proportion to apply to the base saturation flow rate for use in all analyses performed within the jurisdiction. The HCM suggests this calibration be performed every few years or with evidence of driver behavior changes.

Calibration of base saturation flow rate for local conditions is recommended for accurate results within this procedure since these rates can vary dramatically by jurisdiction and largely affect signalized intersection results. For example, larger cities typically have base rates well over 2,000, while smaller towns can be well under 1,600—significantly changing the basis for capacity, which is ultimately used to compute delay and LOS (HCM pp. 31-106 and 31-109).

Saturation Flow Rate Adjustments: There are 11 adjustments to the base saturation flow rate to account for prevailing conditions that, together with effective green time, define capacity by movement. Much of the signalized intersection methodology revolves around saturation flow rate, and many aspects of the analysis are translated into adjustment factor for the saturation flow rates. The adjustments are cumulative in generating the adjusted saturation flow rate, so each adjustment should be understood and scrutinized to best replicate real-world conditions.

- Lane Width – Widths from 10.0 to 12.0 feet receive no adjustment—a change from the HCM 2000, which called for reductions at a width of 11 ft and less and increases at a width of 13 ft and more. This difference could affect any comparisons with older analyses. (Default 12.0 / Typical 12.0 / Range 8.0–16.0)

- Heavy Vehicles and Grade – Replacing the heavy vehicle and grade adjustments with a combined factor, this adjustment accounts for the synergistic effects of heavy vehicles combined with grades without a passenger-car equivalent value used. Equations are provided for negative and positive grades separately.

- Parking – On-street parking is considered if it is within 250 ft of the stop line. As the number of parking maneuvers increases, the saturation flow rate decreases even further. The default maneuver time is 18 s for on parallel parking. This time should be decreased substantially for angle parking. (Default 0 / Typical 8–32 / Range 0–180). Note that even with zero maneuvers per hour, the saturation flow rate will still decrease by 10 percent because of the perceived friction created by the chance of a door opening or a car pulling out. (HCM equation 19-11)

- Buses – Buses that stop within 250 feet of the intersection, near the side or far side, are considered stopping buses. If they do not stop, they are modeled as heavy vehicles, but they are never modeled as both. The default bus stop time is 14.4 s and should be modified if there is any information from the field (large numbers getting on and off, bike racks, wheelchair lifts, etc.) that would indicate the average time is longer. (HCM equation 19-12)

- Area Type – This adjustment is intended to account for the unusual geometry, pedestrian traffic, or additional distractions (double parking, jaywalking, etc.) that are common in a city's central business district (CBD), whether or not the intersection is actually within the boundaries of the CBD. A college campus is a good example of an area that could have these characteristics without being near the center of a city. (HCM p. 19-47)

- Lane Utilization – The HCM assumes that the lane distribution in a multilane group is unequal and that the saturation flow rate will be reduced because both lanes are not typically fully utilized. The reduction is increased where evidence from field observation suggests vehicles congregate in one lane to pre-position themselves for an anticipated move downstream, typically a lane drop, freeway on-ramp, or major generator; these cases can require major adjustments. The volume in the heaviest lane of the multilane group is used to determine this adjustment—even an estimate can be much better than the default values when these situations exist. (HCM equation 19-7 and exhibit 19-15)

- Right Turns – Right-turning vehicles have higher average headway times to navigate the tight radius of the turning movement. This adjustment uses a default value for PCE equal to 1.18, resulting in a heavy vehicle saturation flow adjustment factor of 0.847, reducing the saturation flow rate to about 15 percent. The default assumes a turn radius of about 32 feet and should be adjusted for non-standard designs, like skewed intersections. HCM exhibit 22-23 provides a table to generate this adjustment as a function of turn radius (if known), which can significantly affect the saturation flow rate. The PCE can be computed from this table by dividing the adjustment into 1.00. It can then be used in the adjustment equation. (HCM equation 19-13)

- Left Turns – Left-turning vehicles have higher average headway times to navigate the turning movement's radius. This adjustment uses a default value for passenger-car equivalents equal to 1.05, resulting in a factor of 0.952, reducing the saturation flow rate to about 5 percent. The default assumes a turn radius of about 112 ft and should be adjusted for nonstandard designs, like skewed intersections. HCM exhibit 22-23 provides a table to generate this adjustment as a function of turn radius (if known), which can significantly affect the rate. The passenger-car equivalent can be computed from this table by dividing the adjustment into 1.00. It can then be used in the adjustment equation.

  With the popularity of access management techniques, U-turns have increased at signalized intersections. In order to model the effects of U-turns within the left-turn lane, a passenger-car equivalent of 1.25 can be used in generating a volume-weighted average with left-turns to compute the overall equivalent for the lane group for a more representative adjustment to the saturation flow rate. (HCM equation 19-14)

- Pedestrians – Pedestrians can conflict with permitted left- and right-turning vehicles, which requires adjusting the saturation flow to account for increased headway times. Pedestrian counts for all approaches must be included in the analysis if this conflict is considered significant. (HCM chapter 31 section 2)

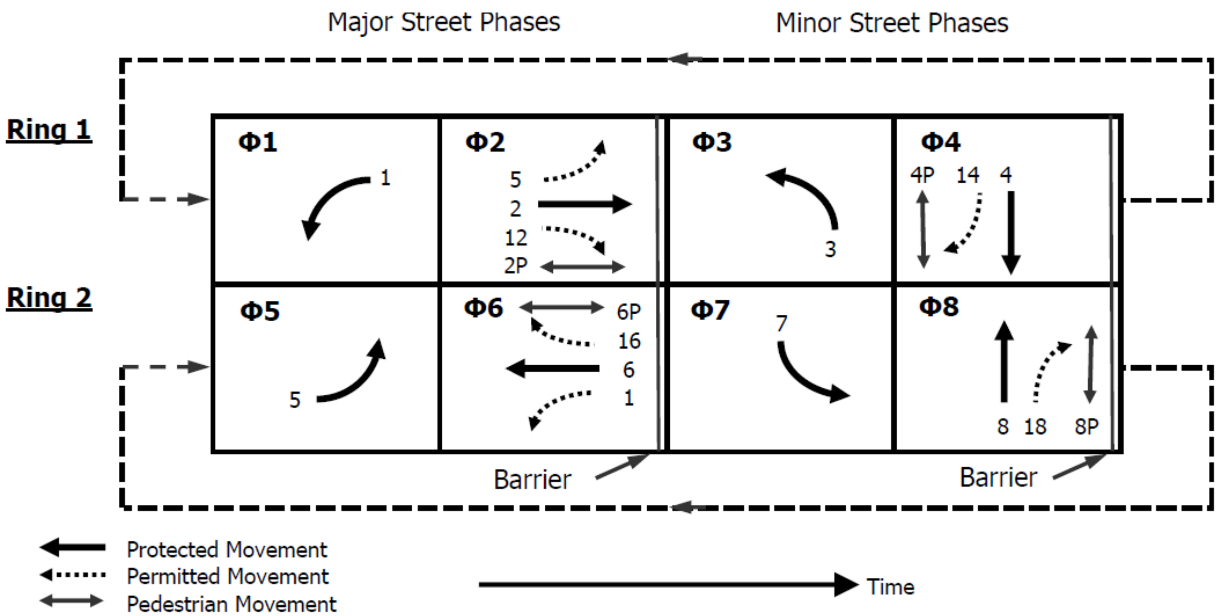
- Bicycles – Bicycles can conflict with right-turning vehicles, which requires adjusting the saturation flow to account for the increased headway times. Bicycle counts for all approaches must be included in the analysis if this conflict is considered significant. (HCM chapter 31 section 2)

- Work Zones – The total approach width while the work zone is active is used in conjunction with the number of left and through lanes open with and without the work zone to develop an adjustment to saturation flow to model the effects of work zone activity. Care must be taken to modify signal timing, detection operation, lane widths, and storage lengths if these change during the active work zone. (HCM equations 31-89 through 31-91)

Care must be taken to modify signal timing, detection operation, lane widths, and storage lengths if these change during the active work zone period.

Signal Phasing: This procedure follows the NEMA standard in defining available signal phases, which extend to include permitted left turns, right-turn overlaps, lead-lag, and Dallas phasing. As shown in figure 13, major street through phases are assigned numbers 2 and 6, with left-turn phases being assigned 1 and 5 by direction. Side-street through phases are assigned numbers 4 and 8, with left-turn phases being assigned 3 and 7, again by direction. Through phases must be designated as allowing permitted left turns or not. Protected left-turn phases can be leading (before the adjacent through phases) or lagging (after the adjacent through phases). Lead-lag phasing occurs when the left-turn phases for one direction lead and lags for the other direction in protected-only mode.

Note that lead-lag phasing, in combination with protected-permitted phasing, must be designated as flashing yellow arrow (FYA) phasing (or Dallas phasing) to eliminate the left-turn trap. Left-turn phases can include right-turn overlaps only if the cross street has exclusive right-turn lanes.



Source: HCM exhibit 19-2.

Figure 13. Illustration. NEMA phasing dual-ring structure with illustrative movements.

Phase Duration: A complex and iterative model is used to estimate the duration of each actuated phase under defined conditions. Knowing the vehicle arrival rate and duration of the red time for a given phase, the queue at the beginning of green can be estimated to predict the green time necessary to process the queue. Realizing that the red time for conflicting movements depends on the green time and the cycle length, this becomes an iterative procedure that accounts for all phases in the cycle. Ultimately, it is this phase duration that is used to generate effective green time to determine the green-to-cycle (g/C) ratio that converts saturation flow rate to capacity.

Note that the phase duration results are average times over the 15-min AP and are not necessarily reasonable if viewed as for a given cycle.

The phase duration model can be overridden if green times are measured in the field or retrieved from a signal system that collects this information and can be accessed. The phase duration model should be used in most analyses, since rarely is field data acquired for average phase times. (HCM pp. 19-12 through 19-14 and 31-2 through 31-22)

Detailed Signal Control Parameters

- Effective Green Time – Phase duration must be adjusted to account for the start-up lost time, clearance lost time, queue service time, green extension time, and extension of effective green components to compute effective green time.

- Start-Up Lost Time – This time (usually taken as 2 s) accounts for the time lost as vehicles in queue accelerate to the saturation flow rate from a stop condition. Normally, this affects the first four to six vehicles. This value has increased due to driver distraction at the stop bar and in the queue due to the use of cellphones and on-board equipment.

- Clearance Lost Time – This time is necessary to clear one movement or direction of travel from the intersection before allowing the subsequent movement or direction to proceed (following the extension of effective green time).

- Queue Service Time – This is the time required to process the vehicles in queue at the beginning of the green.

- Green Extension Time / Passage Time – This is the time required when the green is extended to process vehicles arriving after the queue has been served but before the passage time has been reached.

- Extension of Effective Green – This is the amount of yellow time used as green time due to vehicles entering the intersection during the yellow phase. A default value of 2 s is suggested. This value will increase as the demand approaches and exceeds capacity due to the increased aggression the associated delay generates among drivers.

Back of Queue: This value is computed for percentile averages that range from 50th to 98th percentile options and is expressed in vehicles per lane.

Queue-Storage Ratio: The maximum back of queue is divided by the provided storage length to generate this ratio. Values greater than 1.0 represent queue spillover for turn lanes and queue spillback for through lanes.

CAV Analysis: The HCM 7th Edition incorporated a new method to evaluate CAVs at signalized intersections as a function of the percentage of CAVs in the traffic mix. The following aspects are considered:

- Adjusted base saturation flow rates for the through movements.
- Adjustment factor for the saturation flow rate for protected left-turns and the protected phase of protected/permitted left-turns.
- Adjustment factor for the saturation flow rate for protected left-turns and the protected portion of protected/permitted left-turns.
- Adjustment factors for the saturation flow rate for permitted left-turns and the permitted portion of protected/permitted left-turns.

REVIEW CHECKLIST

Table 15 lists aspects the reviewer should look for in the HCM analysis.

Input variables should comply with other applicable references and guides (e.g., MUTCD) to provide an acceptable level of service.

Table 15. Checklist for signalized intersections.

Signalized Intersections Topics	Type	Default	✓
Lane groups	Input	No default/User must provide	
Turn bay length	Input	No default/User must provide	
Arrival demand	Input	No default/User must provide	
Right-turn-on-red flow rate	Input	No default/User must provide	
Peak hour factor	Input	Vol >1,00 veh/h: 0.92; Otherwise: 0.90	
Percentage of trucks	Input	3%	
Multiple-period analysis	Modeling	See discussion	
Unsignalized movements	Modeling	See discussion	
Area type (CBD, other)	Input	No default/User must provide	
Base saturation flow rate (pc/h/ln)	Input	Metro area: 1,900; otherwise: 1,750	
Saturation flow rate adjustments: Average lane width	Calibration Input	 No default/User must provide	
Approach grade (%)	Input	Flat: 0%; Moderate: 3%; step: 6%	
On-street parking maneuver rate	Input	No default/User must provide	
Pedestrian flow rate (p/h)	Input	No default/User must provide	
Bicycle flow rate (bicycles/h)	Input	No default/User must provide	
Local bus stopping rate (buses/h)	Input	No default/User must provide	
CAV effects (CAV market penetration)	Input	0%	
Signal operations type	Modeling	No default/User must provide	
Detector length (for actuated)	Input	40 ft	
Signal phasing and timing	Input	No default/User must provide	

Signalized Intersections Topics	Type	Default	✓
Cycle length	Input	Se discussion	
Left-turn operational mode (protected/permitted)	Input	No default/User must provide	
Yellow change + red clearance	Input	4 s	
Maximum green	Input	Major street through: 50 s; Minor-street through: 30 s; Left-turn: 20 s	
Minimum green	Input	Major street through: 10 s; Minor-street through: 8 s; Left-turn: 6 s	
Passage time	Input	2 s	
Walk time	Input	7 s	
Pedestrian clearance	Input	Based on 3.5 ft/s walk speed	
Phase recall	Input	No recall	
Offsets (coordinated arterials)	Input	Travel time in phase 2	
Detailed signal control parameters	Calibration	See discussion	
Unsignalized movements	Modeling	See discussion	
Movement control delay	**Service Measure**	-	
Back of queue	MOE	-	
Queue-storage ratio	MOE	-	
Critical v/c ratio	MOE	-	
Back of queue	MOE	-	
Approach and intersection delay	MOE	-	

HCM LIMITATIONS

This methodology should not be used to analyze interchange intersections directly; instead, use chapters 23 and 34. In addition to the above conditions, the HCM methodology does not directly account for the following:

- 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.
- 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.
- Storage of shared-lane left-turning vehicles within the intersection to permit through vehicles to bypass in the same lane.
- Rest-in-walk mode for actuated and noncoordinated phases.
- Preemption or priority modes.
- Phase overlap: right turn movements may overlap exclusive left-turn movement phases from the opposite direction).
- Gap reduction or variable initial settings for actuated phases.

CHAPTER 11. URBAN STREET SEGMENTS AND FACILITIES - HCM CHAPTERS 16, 17, 18, 29, AND 30

VARIABLES AND MODEL ASPECTS

LOS: The average travel speed of through vehicles now determines LOS rather than the average travel speed as a percentage of base FFS. The threshold for LOS A changed from 85 percent BFFS to 80 percent BFFS. Other LOS results could change for the performance measure boundaries because of new units and rounding. (HCM exhibit 18-1)

Flow Profile: Multiple signals along an arterial can now be modeled using the flow profile to estimate the proportion of vehicles arriving on green. A platoon dispersion model is included that considers running time and access point flows to predict the arrival flow rate at the downstream signal. The inclusion of this model greatly improves the computation of uniform delay for the through movement at each signal that is used in the determination of travel speed.

The flow profile must be allowed to compute the proportion of vehicles arriving on green whenever possible to include the upstream signal in the analysis. Overriding this analysis by inputting the arrival type is seldom justified. For example, an arrival type of 4 uses a proportion of vehicles arriving on green of 1.33 (which could be other values between 1.00 and 1.67) as only a gross estimate. An arrival type of 5 with a g/C ratio of 0.6 (not uncommon) generates a uniform delay of zero (not defendable). (HCM exhibit 18-14, equation 18-9, and chapter 30 section 3)

Note: Side-street approaches are very rarely coordinated with the major-street signals, so these types of arrival values would almost always be 3 to represent random arrivals.

Lane Blockage: This procedure is used to adjust the saturation flow rate of the movements entering a segment when one or more downstream lanes are blocked. The calculation sequence begins with an estimate of the capacity for each traffic movement discharged to the downstream segment, then the capacity of the downstream segment as influenced by the midsegment lane restriction is computed, and the two values are compared. In the event the movement capacity exceeds the downstream segment capacity, the movement saturation flow rate is reduced proportionally using an adjustment factor for downstream lane blockage, which is computed for each movement entering the subject segment (HCM equations 30-29 and 30-30).

Sustained Spillback: The adjustment factor for sustained spillback is used to evaluate the effect of spillback from the downstream intersection, quantified as a reduction in the saturation flow rate of upstream lane groups entering the segment. The calculation of the adjustment factor for spillback is one part of the urban streets procedure. (HCM chapter 29 section 3)

Access Points: Flow rates from access points between signalized intersections are used within the flow profile to estimate decay effects on the platoon and the proportion of vehicles arriving on green. Delay due to left- and right-turning vehicles at access points is also used in the computation of running time that affects travel speed.

Note that while collecting data for access points may be costly, the effects on the model can be significant in terms of generating accurate results. Flows in and out of side streets and driveways can easily affect speeds between signals and arrival rates at signals enough to change the running time and approach delay, creating differences in LOS values. (HCM equation 18-13 and chapter 30 section 4)

For segments that include access points between signals with flow rates that can significantly affect demand balancing, travel speed, and platoon integrity on the segment, turning movement data from the access points should be collected to account for the access-point effects on the street segment. For example, several side streets or driveways (or fewer with higher volumes) can reduce travel speed and the proportion of vehicles arriving on green enough to change the LOS for the segment.

Upstream Filtering: Computing the upstream filtering is part of the urban street facility method, since adjacent signal information is known, which overcomes the potential misuse of this very sensitive parameter.

It is rarely justifiable to override this value if data from the upstream signal is available. The value can even be computed for minor street approaches by separately modeling those upstream signals. For example, arbitrarily changing the default value of 1.0 to 0.1 can lower the delay value by 10–20 seconds for movements with v/c ratios greater than 0.0. A value of 1.0 should be used when upstream data are not known. (HCM equation 19-6)

Flow Balancing: Since turning movement count data are usually collected at each signalized intersection on different days, the flows among the intersections are generally not balanced. For the evaluation to work correctly, these inconsistencies must be resolved so balanced flows can be used in the models. This adjustment is reflected in the adjusted flow rates in combination with the shared lane model used for signalized intersections. (HCM pp. 18-25 and 18-26 and exhibits 18-9 and 18-10)

RTOR Balancing: RTOR volumes have now been incorporated into the flow profile process to account for these movements as they affect platoons and the proportion of traffic flow arriving on green. Including RTOR flow is highly desirable to overcome the elimination of these flows in the signalized intersection procedure. (HCM pp. 30-3 through 30-5)

BFFS: This equation now has two additional terms to calibrate for local conditions and to account for parking activity along the segment. (HCM equations 18-3)

Calibration Factor: This factor now permits the adjustment of BFFS if field data are available and can be applied for overall local conditions or specific street types. A procedure for measuring FFS in the field is available in HCM chapter 30. (HCM pp. 18-28, 18-29, 30-41, and 30-42)

Parking Activity: This adjustment factor for on-street parking has been added to the BFFS equation. This factor is a function of the proportion of the link length with on-street parking on the right side. (HCM exhibit 18-11)

FFS: Computed as an adjustment to BFFS, this value tends to be higher than the speed limit. (HCM equation 18-5)

Arrival Type: With the implementation of the flow profile, arrival type is not used to compute the proportion of vehicles arriving on green when analyzing multiple signalized intersections on an urban street, with the exception of boundary or side-street approaches; in these cases, information about the upstream signal is not within the scope of the analysis. (HCM pp. 18-32 through 18-34)

Optimizing Timing: Cycle length, splits, and offsets are considered in the HCM procedures and make a significant difference in both the operation of the arterial and the analysis results. While the HCM does not define models for optimized signal timing, guidance is provided on the use of alternative tools. For example, signal and arterial optimization can be accomplished using a generic optimization algorithm on several objective functions to minimize delay, stops, or travel time or to maximize speed or percentage BFFS for the best LOS. (HCM chapter 29 section 4)

Roundabout Corridors: This methodology provides for analyzing urban street segments bounded by roundabouts. The basis of the approach is to compute average travel speed to generate LOS using the urban streets procedures and incorporating adjustments for roundabouts as boundary intersections. (HCM chapter 30 section 9)

- *BFFS*: This parameter is computed exactly the same for segments bounded by roundabouts and signalized intersections. (HCM equation 30-72 and exhibit 30-43)

- *Geometric Delay*: New data requirements include the average width of circulating lanes and the largest inscribed circle diameter. These data are used to generate the central island diameter, average radius of the through circulating path, circulating speed, and subsegement lengths. (HCM exhibits 30-40 through 30-42)

- *FFS*: FFSs for Subsegment 1 and Subsegement 2 are computed as functions of the influence areas and may be lower than the speed limit (unlike segments bounded by signalized intersections). FFSs for segments without roundabout influence are computed exactly the same as for segments bounded by signalized intersections for purposes of comparison with the subsegment values in which the minimum of the three is used to compute running time. (HCM equations 30-73 through 30-86)

- *Running Time*: This equation is modified for yield control at roundabouts with a start-up lost time of 2.5 (not 2) s and limiting the first term to the v/c ratio with a maximum value of 1.00. (HCM equation 30-87)

- *Control Delay*: The control delay of the entering lane(s) is computed using the roundabout procedure, proportioning the delay in each lane (if more than one) by the through-flow rate. (HCM chapter 22, equations 30-88 and 22-17)

- *Geometric Delay*: The segment geometric delay is computed for each subsegment as a function of FFS and circulating speed within the inscribed circle diameter. (HCM equations 30-89 and 30-90)

Through Delay: Delay for the through movement is the sum of the approach control delay and the subsegment geometric delays. (HCM equation 30-91)

Travel Speed and LOS: Ultimately, travel speed is calculated to determine LOS exactly as for segments bounded by signalized intersections. (HCM equation 18-15 and exhibit 18-1)

Travel Time Reliability: This methodology generates a distribution of trip travel time over an extended period as affected by variations in demand, weather, work zones, incidents, and special events on an urban street facility. (HCM chapter 17)

Base Data Set: Intersections, segments, and periods are defined in a complete urban streets analysis as the basis for the distributed generation of scenarios. (HCM p. 17-12)

Demand: Distribution of values by the time of the day, day of the month, and month of the year. (HCM pp. 17-15 through 17-17 and 17-23)

Weather: Nearest city for the provided database is selected for the most appropriate distribution of weather events by month for precipitation, snowfall, and temperature variations. (HCM pp. 17-17 and 17-23)

Incidents: Types, locations, and severity proportions are provided in terms of frequency, response times, and clearance times. (HCM pp. 17-18 through 17-22 and 17-23 through 17-24)

Special Events: Defines specific times and effects on demand. (HCM pp. 17-22 and 17-24)

Work Zones: Defines specific project locations, times, durations, work zone modifications. (HCM pp. 17-22 and 17-24)

Scenario Generation: Based on the desired number of periods, unique combinations of demand, capacity, geometry, and traffic control conditions are produced to provide the distribution of results from which to compute the analysis parameters for describing travel time reliability. (HCM p. 17-26 and chapter 29 section 2)

TTI: TTI is defined as the ratio of the actual travel time on a facility to the travel time at the base free-flow speed. (HCM pp. 17-9 through 17-10 and 17-28 through 17-30)

PTTI: PTTI is defined as the ratio of the 95th percentile highest travel time to the travel time at the base free-flow speed. (HCM pp. 17-10 and 17-30)

ATDM: ATDM tactics can be evaluated by adapting the facility configuration and controls to react to variations in demand, weather, and incidents. These might include changes to speed and signal control (e.g., adaptive signal timing, priority treatments), modifications to geometric configurations (e.g., reversible lanes, dynamic lanes, turn-lane assignments), or combinations thereof. (HCM chapter 17 section 4)

REVIEW CHECKLIST

Table 16 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 16. Checklist for urban street segments.

Urban Streets Topics	Type	Default	✓
Flow profile	Input/ Intermediate variable	No default/User must provide	
Access points	Modeling	No default/User must provide	
Flow balancing	Modeling	See discussion	
Segment length	Input	No default/User must provide	
Restrictive median length	Input	No default/User must provide	
Parking activity	Input	No default/User must provide	
Speed limit	Input	No default/User must provide	
Work zones	Input	No default/User must provide	
Incidents	Input	No default/User must provide	
Arrival type	Intermediate variable/calibration	-	
Upstream filtering	Intermediate variable/calibration	-	
Lane blockage	Intermediate variable	-	
Sustained spillback	Intermediate variable	-	
Average speed	**Service measure**	-	
Optimizing timing	Modeling	See discussion	
Travel time reliability	MOE	-	
ATDM	Modeling	-	

HCM LIMITATIONS

The methodology in this chapter does not take into account, nor is it applicable to (without modification by the analyst), cases involving:

- 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 affecting 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.
- Cross-street congestion or a railroad crossing that blocks through traffic.

CHAPTER 12. RAMPS TERMINALS AND ALTERNATIVE INTERSECTIONS – HCM CHAPTERS 23 AND 34

HCM chapter 23 discusses both interchange ramp terminals and alternative intersections because they combine multiple intersections in a cluster. Due to the close spacing between these intersections, they are operationally interdependent and must be analyzed as a single unit for more accurate results. Nevertheless, the methodology discussed in this chapter strongly relies on methods used for individual intersections, with specific adjustments to account for the interdependent operation of ramp terminals and alternative intersections. (HCM chapters 19 through 22)

The HCM methodology in this chapter is organized into three distinct parts, as follows:

- Part A: overview of concepts common to both alternative intersections and interchanges.
- Part B: evaluation of surface street-freeway interchanges.
- Part C: evaluation of alternative intersections.

PART A: COMMON CONCEPTS

VARIABLES AND MODEL ASPECTS

Alternative Intersections and Interchanges: "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. (HCM p. 23-1)

Note: Lane groups and intersections are not considered in LOS except to check v/c and queue-storage ratios. (HCM exhibits 23-10 and 23-12)

A comparison of movement delay from each intersection to the sums used for LOS can reveal interactive issues. For example, the series of O-D delays could generate acceptable levels of service when compared to the interchange ramp terminal thresholds but be less acceptable when scrutinized by individual movements compared to the signalized intersection thresholds.

Experienced Travel Time (ETT): For these distributed intersections, each O-D path can include extra distance travel time (EDTT) in addition to control delay at each intersection that must be included in the analysis for unbiased comparison purposes. For this reason, the user must provide additional geometric information to be able to compute these results correctly, including the extra distance traveled along the ramp and the design speed of the ramp. The EDTT value can be negative for right turns because of the destination heading away from the freeway centerline, creating net savings in the distance traveled. (HCM equations 23-1 through 23-10 and exhibits 23-6 through 23-9, 23-11, and 23-12)

Signalized Intersections: Normally, two signalized intersections that interact as interchange ramp terminals are modeled together to generate origin-destination results. Delay is computed for all movements then combined into origin-destination pairs for defining LOS. Several factors affecting saturation flow rates and effective green times are modified for signalized intersections that are part of interchanges. (HCM pp. 23-5 and 23-6)

PART B: INTERCHANGES

Lane Utilization: More complete models for lane utilization adjustment to saturation flow rates for external approaches (using information from the downstream signals) are implemented. For internal approaches and those with more than four through lanes, chapter 19 default values are used.

Note: Default values should be overridden with heaviest lane volumes when conditions are not typical and warrant collecting and using these data. (HCM equation 23-16)

Saturation Flow: Adding traffic pressure and turn radius to enhance further the saturation flow rate adjustment can be critical to results.

Note: Turn-radius equivalencies can be useful in the analyses (as calculated in chapter 19) for skewed intersections or other nonstandard designs. (HCM equations 23-15 and 23-19 through 23-23, and exhibits 23-23, and 23-27)

Downstream Queue: If a downstream (internal link) queue exists (as computed by the chapter 19 methodology) that would inhibit movement from the upstream signal, additional lost time is incurred and accounted for by this procedure. (HCM exhibit 23-28 and equations 23-29 through 23-34)

Demand Starvation: If there is no queue present at the downstream approach and no arrivals from the upstream signal during the green, additional lost time is incurred and accounted for by this procedure. Demand starvation is more likely to occur on segments with short length (less than 700 ft) and poor signal progression. (HCM exhibit 23-28 and equations 23-38 and 23-39)

Effective Green Time: When either a downstream queue or demand starvation occurs, the effective green time is decreased, reducing capacity and increasing delay for the affected movement. (HCM equations 23-24 through 23-28)

Diverging Diamond Interchanges (DDI): 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.

PART C: ALTERNATIVE INTERSECTIONS

Median U-Turn (MUT) intersections: At-grade intersections at which major- and minor-street left-turn movements are rerouted. Minor-street through movements are not rerouted. (HCM p. 23-5)

Restricted Crossing U-Turn (RCUT) intersections: At-grade intersections at which minor-street left-turn and through movements are rerouted. Major-street left-turn movements are not rerouted. (HCM p. 23-5)

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. (HCM p. 23-5)

AS a note, DLT, RCUT, and MUT intersection analyses begin with demand data for the conventional signalized intersection, which is distributed to the supplemental intersections to population those turning movements appropriately for the overall OD values.

REVIEW CHECKLIST

Table 17 serves as a checklist that the reviewer should use in the HCM analysis in addition to those parameters used for analysis of conventional unsignalized or signalized intersections.

Table 17. Checklist for interchange ramp terminals.

Interchange Ramp Terminals Topics	Type	✓
Alternative intersections and interchanges	**Modeling**	
Experienced travel time (ETT)	**Service measure**	
Signalized vs unsignalized Intersections	Modeling	
lane utilization	Input/calibration	
saturation flow	Calibration	
downstream queue	MOE	
demand starvation	Intermediate variable	
effective green time	Intermediate variable	

HCM LIMITATIONS

The methodology in this chapter does not take into account, nor is it applicable to (without modification by the analyst), cases involving:

- Oversaturated conditions, particularly when the downstream queue spills back into the upstream intersection for long periods.
- 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 link travel times and speeds (the methodology does provide delay estimates).
- 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).

CHAPTER 13. TWO-WAY STOP CONTROL (TWSC) – HCM CHAPTERS 20 AND 32

Two-way stop-controlled (TWSC) intersections are unsignalized intersections at which drivers on the major street have priority over drivers on the minor-street approach(es). Drivers on minor streets must stop before entering the intersection. Drivers turning left 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. Movements are ranked in order of relative priority. The methodology revolves around the concept that lower-rank movements must find gaps in traffic with higher-ranked movements to clear the intersection. The ranks are as follows:

- Rank 1: Mains street through and right-turn movements.
- Rank 2: Main street left-turn (and U-turn) movements and minor street right-turn movements.
- Rank 3: Minor street through movements.
- Rank 4: Minor street left-turn movements.

VARIABLES AND MODEL ASPECTS

Two-Stage Gap Acceptance: A raised, striped or TWLTL often causes a special gap acceptance phenomenon known as "two-stage gap acceptance," in which a significant proportion of minor-street drivers cross half of the major street and then pause in the median to wait for a gap on the other approach. When median storage exists, the user needs to define the number of vehicles that can be stored in the median waiting to cross the opposite direction major street (stage II crossing).

Lane Configuration: Lane configuration needs to be entered for each approach. Approach lanes can be assigned for exclusive movements, or shared lanes may be used. The conflicting volumes for all movements are highly dependent on lane configuration and volumes.

Arrival Demand: Counting vehicles as they cross the stop line is not adequate for collecting data to analyze congested conditions. If demand approaches or exceeds capacity, arrival rate must be known to incorporate arrival demand in this methodology. Arrival demand is calculated by collecting arrival data upstream of all queues associated with the approach and then reconciling the approach rate to each movement at the stop line.

PHF: As with other methods in HCM, a 15-min AP within the analysis hour should be used for TWSC methods. One PHF is used for the intersection.

Grades and Heavy Vehicles: Approach grades and percentage of heavy vehicles are used to calculate default critical headway and follow-up headway for left-turn movements on the main street and all movements on the minor street.

Flared Minor Street Right: A flared geometry for the minor-street approach may be used to provide some storage area for right-turning vehicles in shared through and right-turn lanes.

Calibration: Two sets of calibration parameters can be used for TWSC analysis adjustment.

- $f_{c,x,y}$. Each movement at a TWSC intersection faces a different set of conflicts directly related to the nature of the subject movement. HCM 7th Edition 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 (veh/h) that conflicts with movement x as those sensitive to conflicting flow factors $f_{c,x,y}$, which represent the weight of each conflicting movement y relative to the analyzed movement x. Values of $f_{c,x,y}$ can be calibrated based on the professional judgment of the analyst and agencies for particular conflicting movements of the studied intersections. In the computation of $v_{c,x}$, there are default conflicting flow factors, $f_{c,x,y}$. If desired, the user may calibrate these values.

- Critical headway and follow-up headway times should be calibrated to local conditions for accurate results within this procedure, since area population, traffic level, and approach speed can all significantly affect the gap acceptance by drivers. The process for collecting critical and follow-up headway data in the field is quite complex, but measuring delay is relatively simple (following the procedure outlined on HCM pp. 31-99 through 31-105). Once delay is known, the critical headway and follow-up headway times can be estimated as those that will generate the field-measured delay as computed using the methodology.

A "reality check" of delay and queue can reveal the need to calibrate critical and follow-up headway values, since the HCM defaults are quite conservative and can yield higher delays and longer queues than are reasonable for a given location—especially with greater demand. For example, when traffic levels are high (e.g., during peak periods) or drivers are aggressive (e.g., in larger cities) the default values can yield much greater delay and queueing than really exist because drivers will accept much shorter gaps.

Saturated Flow Rate: saturated flow rate for the main road is entered to estimate the capacity of the main road through and right turns, which will affect the crossing opportunities for other movements.

Upstream Signals: The effects of upstream signals on conflicting flow rates are modeled using the proportion of time-blocked results from an urban streets analysis. (HCM equations 20-33, 20-34, and 20-35)

Pedestrian Crossing Volumes: Pedestrian volumes are used not only for the Pedestrian LOS mode, but also are added to the conflicting flow for motorized movements that yield to crossing pedestrians.

Pedestrian Crossing Treatments: A series of pedestrian crossing treatments are analyzed to generate motorist yield rates to be used in calculating adjustments to pedestrian delay and LOS, as shown in table 18.

Table 18. Pedestrian crossing treatments and yield rate.

Crossing Treatment	Yield Rate (%)		Sample Size
	Average	Range	
No treatment (unmarked)	24	0-100	37
Crosswalk markings only (any type)	33	0-95	58
Crosswalk markings, plus:			
Pedestal-mounted flashing beacon	26	0-52	2
Overhead sign	35	12-57	2
Overhead flashing beacon (push-button)	51	13-91	14
Overhead flashing beacon (passive)	73	61-76	29
In-roadway warning lights	58	53-65	11
Median refuge islands	60	0-100	21
Pedestrian crossing flags	74	72-80	6
In-street pedestrian crossing signs	76	35-88	20
Rectangular rapid-flashing beacon (RRFB)	82	31-100	64
School crossing guard	86	-	1
School crossing guard and RRFB	92	-	1
Pedestrian hybrid beacon (HAWK)	91	73-99	37
Mid-block crossing signals (half signals)	98	94-100	13

Control Delay: Service measure for unsignalized intersections, the delay quantifies the increase in travel time due to stop-sign control. It is also a surrogate measure of driver discomfort and fuel consumption.

Rank 1 Delay: Delay to major-street through and right-turning vehicles (Rank 1 movements) should be considered when there is no exclusive major-street left-turn lane (or when the left-turn lane is inadequate for the left-turn queue) as a potential design component and as part of overall intersection delay for comparison purposes. Short storage left-turn lanes and left-turn pockets may be used to store vehicles and avoid delays to Rank 1 movements. (HCM exhibit 20-15 and equations 20-43 and 20-44)

Level of Service: While average control delay is used to determine level of service in all intersection analyses, thresholds differ between signalized and unsignalized control. This presents a dilemma when comparing delay between these control types.

Queuing: While the 95th percentile queue parameter is computed as part of the procedure, the average queue is equivalent to vehicle hours of delay for any lane.

REVIEW CHECKLIST

Table 19 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 19. Checklist for two-way stop-controlled intersections.

Two-Way Stop Control Topics	Type	Default	✓
Undivided or divided major street with median storage (two-stage crossing)	Modeling	No default/User must provide	
Median storage area	Modeling	No default/User must provide	
Lane configuration	Input	No default/User must provide	
Arrival demand per movement	Input	No default/User must provide	
Intersection-wide peak hour factor	Input	0.92	
Grades and heavy vehicle percentages	Input	0% grade, 3% trucks	
Flared minor-street approach and storage	Input	No default/User must provide	
Critical and follow-up headway	Calibration	Exhibit 20-17	
Conflicting flow factors	Calibration	Exhibits 20-8 to 20-16	
Upstream signals	Input	No default/User must provide	
Pedestrian volumes	Input	No default/User must provide	
Control delay	**Service measure**	-	
Rank 1 delay	MOE	-	
Queuing	MOE	-	

HCM LIMITATIONS

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.

CHAPTER 14. ALL-WAY STOP CONTROL (AWSC) – HCM CHAPTERS 21 AND 32

All-way stop-controlled (AWSC) intersections are common in the United States. In an AWSC intersection, all approaches are controlled with stop signs, and no street has priority. After stopping, all vehicles proceed through the intersection on a first-come-first-served basis.

Flows are determined by a consensus of right-of-way and may depend on local behavior. Giving the priority for the vehicles coming from the right is the standard rule in most areas, but 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, which 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). (HCM p. 21-2)

As drivers observe vehicles at the other approaches, opposing through movements may occur simultaneously, as well as right-turning movements with opposing left-turn movements.

VARIABLES AND MODEL ASPECTS

Lane Configuration: While three-way stop control at T-intersections can be analyzed, intersections with three stop-controlled approaches at a four-leg intersection are not covered by the methodology. Three stop-controlled approaches at a four-leg intersection can be coded as all-way stop-control, then converted to simulation to remove stop control on one approach for analysis.

Arrival Demand: Counting vehicles as they cross the stop line is not adequate for collecting data to analyze congested conditions. If demand approaches or exceeds capacity, arrival rate must be known to use demand in this methodology by collecting arrival data upstream of all queues associated with the approach, then reconciling the approach rate to each movement at the stop line.

PHF: As with other methods in HCM, a 15-min AP in place of one hourly analysis should be used for AWSC methods. One PHF is used for the intersection.

Lane Utilization: Defining the percentage of vehicles in each lane of multiple-lane approaches is the responsibility of the user. When this is unknown, an equal lane distribution can be assumed.

Heavy Vehicle Percentages: Percentage of heavy vehicles is entered for each movement. This value is used in the calculation of headway adjustment.

Delay: A service measure for unsignalized intersections, delay quantifies the increase in travel time due to stop-sign control. It is also a surrogate measure of driver discomfort and fuel consumption.

Queuing: While the 95th Percentile Queue parameter is computed as part of the procedure, the average queue is equivalent to the vehicle hours of delay for any lane. (HCM p. 21-33)

REVIEW CHECKLIST

Table 20 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 20. Checklist for all-way stop control intersections.

All-Way Stop Control Topics	Type	Default	✓
Lane configuration	Modeling	No default/User must provide	
Arrival demand	Input	No default/User must provide	
Intersection peak hour factor	Input	0.92	
Percent of heavy vehicles	Input	3%	
Lane utilization on shared lanes	Input / calibration	Equally divided	
Control delay	**Service measure**	-	
Queuing	MOE	-	

HCM LIMITATIONS

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 departure headway of affected vehicular movements, but the magnitude of this effect is unknown.

CHAPTER 15. ROUNDABOUTS – HCM CHAPTERS 22 AND 33

Roundabouts are intersections with a generally circular shape characterized by the 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.

VARIABLES AND MODEL ASPECTS

Lane Configuration: Lane configuration needs to be entered for each approach. Approach lanes can be assigned for exclusive movements or shared lanes (including bypass lanes) may be used. The number of roundabout circulating lanes is also entered, affecting the roundabout capacity model.

Inscribed Circle: Defined as the diameter of the largest circle that can be inscribed within the outer edges of the circulatory roadway.

Number of Lanes and Lane Width: The number and average width of circulating lanes is measured in the section of circulatory roadway immediately downstream of the entry.

Arrival Demand: Demand is entered by approach for each movement. Collecting turning movement count data can be challenging since entering vehicles must be followed through to their exiting legs to properly consider through movements, left turns, and U-turns as drivers navigate the roundabout.

Percentage of heavy vehicles and equivalent factors: The percentage of heavy vehicles is entered for each movement. This value is used to adjust flow rate for heavy vehicles in conjunction with a passenger car equivalent, which can be calibrated by the user.

Lane Utilization: De-facto turn lanes are assumed for two-lane approaches based on the relative movement demands in relation to the designated lane assignments. Flow rate percentages can be allocated if field data are available.

Bypass Lane Definition: Right-turn bypass lanes are defined as yielding or non-yielding based on their interaction with exiting flow. Yielding right-turn bypass lanes merge at the point of exit, with the exiting flow becoming the conflicting flow for this movement and non-yielding right-turn bypass lanes merge downstream.

Pedestrians: The effect of pedestrians on entering vehicles only applies if the conflicting flow is less than 881 vehicles per hour where queues are not guaranteed. There is nothing in the methodology to account for the effect of pedestrians on exiting vehicular flow, although this could be significant in some situations.

Critical Headway and Follow-up Headway Times: As with the TWSC, these are the main calibration variables to adjust the method to local conditions for accurate results within this procedure, since population, traffic level, and familiarity (over time) can have significant effects on the operation of the roundabout. These parameters should be calibrated using field data.

Geometric Delay: The delay introduced by navigating the circulatory roadway is considered beyond control delay for computing travel speed as functions of the inscribed circle, circulating speed, and segment free-flow speed. Geometric delay is then included with control delay in travel speed determination.

Control Delay: Service measure for unsignalized intersections, the control delay quantifies the increase in travel time due to the roundabout operation characteristics. It is given per lane in the roundabouts method, while an intersection average is also computed.

Queuing: The 95th percentile queue parameter is computed as part of the procedure, as with other intersection methods.

Level of Service: LOS is based on travel speed as a function of base free-flow speed using the urban streets thresholds.

Roundabout Segments: This methodology provides for the analysis of urban street segments bounded by roundabouts. The basis is to compute average travel speed to generate level of service using the Urban Streets procedures with adjustments for roundabouts as boundary intersections. (HCM chapter 30 section 9)

The main inputs affecting roundabout segment analysis are mostly related to FFS estimation as defined in the urban street segments chapter:

- Segment length.
- Number of lanes (calculated based on the number of lanes on the upstream and downstream roundabout approaches).
- Speed limit.
- The base free-flow speed calibration factor, scalib.
- Restrictive median length.
- Right-hand access points.
- On-street parking.
- Mid-segment demand flow.
- Travel speed (determines the LOS computed from the combination of running time along the segment and through delay at the roundabout, converted to speed using the urban streets procedure).

REVIEW CHECKLIST

Table 21 serves as a checklist that the reviewer should look for in the HCM analysis.

Table 21. Checklist for roundabouts.

Roundabouts Topics	Type	Default	✓
Approach and circulating lane configuration	Modeling	No default/User must provide	
Inscribed circle	Input	No default/User must provide	
Circulating lanes	Input	No default/User must provide	
Arrival demands per movement	Input	No default/User must provide	
Percentage of heavy vehicles	Input	3%	
Passenger car equivalent	Calibration	2	
Lane utilization	Input/Calibration	Exhibit 22-9	
Bypass lanes (yielding or non-yielding)	Modeling	No default/User must provide	
Pedestrians	Input	No default/User must provide	
Critical and follow-up headways	Calibration	Page 33-6	
Control delay	**Service measure**		
Queuing			
Roundabout segments	Modeling	No default/User must provide	

HCM LIMITATIONS

The procedures presented in this section cover many of the typical situations a user may encounter in practice. However, alternative tools can produce a more accurate analysis for some applications. 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.

CHAPTER 16. NETWORK ANALYSIS – HCM CHAPTER 38

The HCM is a very effective tool for evaluating the quality of service for different transportation elements, such as freeways, highways, intersections, urban streets, and interchanges. However, prior editions of the HCM lacked a methodology for evaluating networks comprised of multiple elements, such as freeway-urban street interactions.

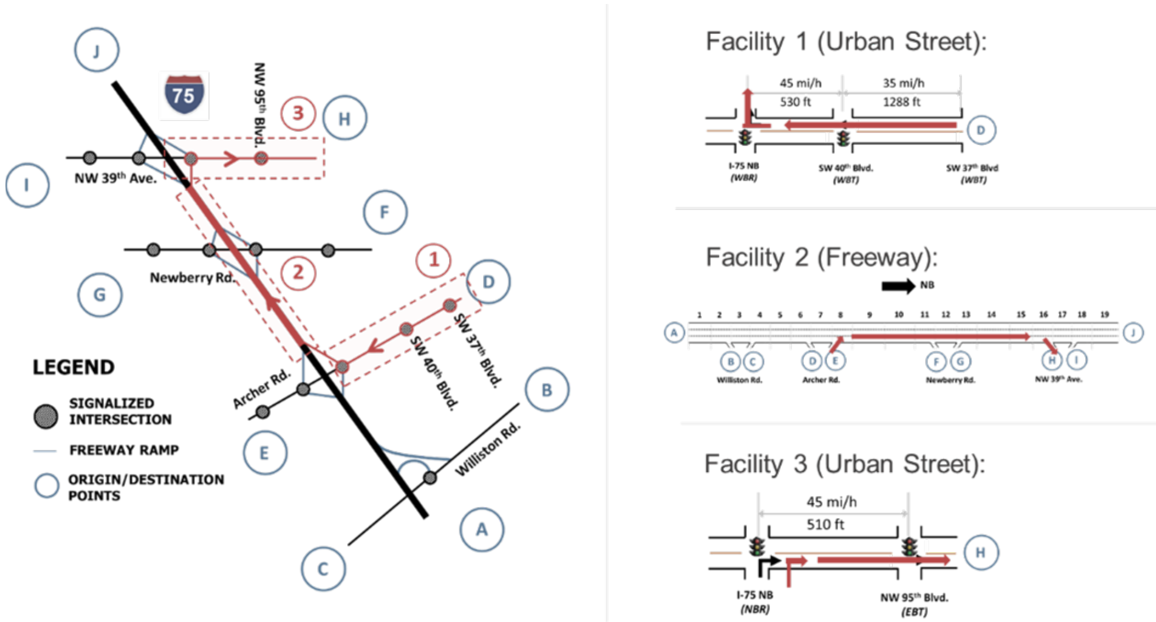
The results of this report were incorporated into the HCM 7th Edition as a new "Chapter 38 – Network Analysis." This section highlights some major features of this new methodology.

COMMON PERFORMANCE MEASURE: TRAVEL TIME

The evaluation of the quality of service for trips over different facility types required moving from segment-based performance measures to OD measures.

Travel time between OD pairs was established as the common performance measure to evaluate systems with freeways and urban streets. This measure is already used in the HCM to analyze urban streets, but additional modifications were required to adapt the methods for freeways, such as implementing models for lane-by-lane analyses.

Figure 1416 illustrates a typical application of the new methodology. Currently, to evaluate a trip between points D and H in a network, the user would need to model urban streets and freeway segments separately, with several limitations and the need for multiple assumptions. The new methods in chapter 38 facilitate this type of analysis and yield travel-time-based performance measures.



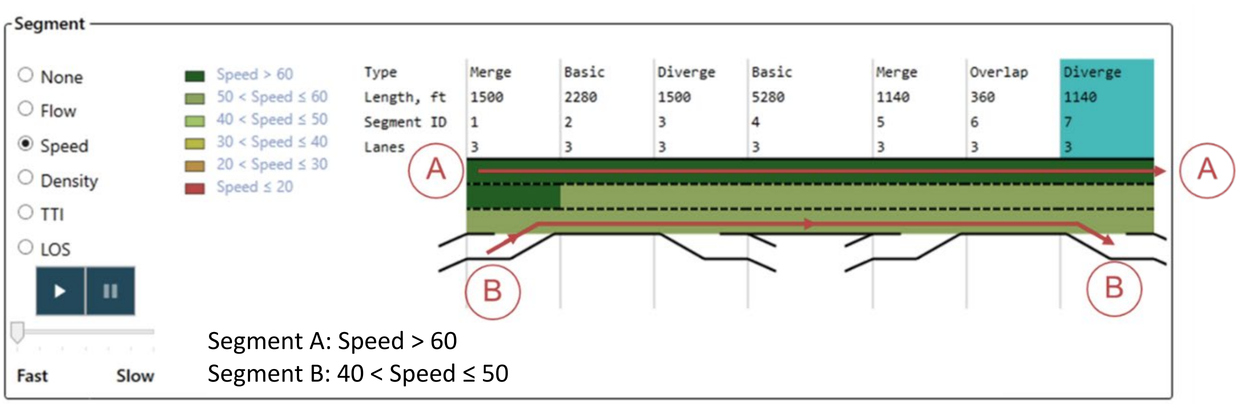
Source: FHWA.

(a) Network Methodology (b) Isolated Facility Methods

Figure 14. Illustrations. Sample network analysis using the new HCM methods.

Freeway Lane-by-Lane Analysis: Freeway speeds and travel times can vary widely depending on the lane used, and each OD pair is likely to use a specific set of lanes over each segment. Therefore, new methods were developed to evaluate the performance of freeway segments on a lane-by-lane basis as a function of factors such as demand-to-capacity ratio, presence of nearby ramps, percentage of heavy vehicles, and grade.

Figure 15 illustrates an application of the OD-oriented travel time analysis. Although routes A-A and B-B traverse the same set of segments, they are expected to use different lanes and therefore yield different travel times.



Source: FHWA.

Figure 15. Illustration. Lane-by-lane travel time analysis.

Queue Spillback Analysis on Freeways: Queue spillback at freeway ramps occurs when one road or traffic control element has insufficient capacity and the queue extends beyond the available storage in the ramp roadway, disrupting the upstream traffic flow.

The new methods evaluate queue spillback through both off-ramps and on-ramps by integrating the analyses of intersections and freeways. Resulting queues from a congested ramp terminal, arterial traffic control, or a downstream congested freeway (for freeway-to-freeway connections) are used as inputs to calculate the effects of queue spillback in the quality of service of a given freeway.

Different methods are available to estimate the impact of queue spillback on signalized intersections, stop-controlled intersections, or roundabouts. Ramp metering can also be an input to the method at on-ramps as it is a frequent source of on-ramp congestion.

REVIEW CHECKLIST

The network level analysis requires attention to all aspects considered in the freeway facilities and urban streets analysis. Table 22 lists network-specific aspects the reviewer should look for in the HCM analysis.

Table 22. Checklist for network analysis.

Network Topics	Type	✓
Mainline freeway and arterial crossings	**Modeling**	
Ramp terminal type	Modeling	
Freeway lane-by-lane analysis	Output	
Od travel time	**Service measure**	
Capacity checks	Outputs	
Queue spillbacks	Output	