Highway Capacity Manual Reference Guide

December 2016

FHWA-HOP-14-037

U.S. Department of Transportation
Federal Highway Administration

Notice

Quality Assurance Statement

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.	
FHWA-HOP-14-037			
4. Title and Subtitle Highway Capacity Manual Review Guide (HCMRG)		**5. Report Date** September 2014 **6. Performing Organization Code:**	
7. Author(s) William M. Sampson (University of Florida McTrans Center)		**8. Performing Organization Report No.**	
9. Performing Organization Name and Address Leidos 11251 Roger Bacon Drive, 3rd Floor Reston, VA 20190 University of Florida McTrans Center PO Box 116585, Gainesville, FL 32611		**10. Work Unit No.** **11. Contract or Grant No.** DTFH61-12-D-00020-T-13015	
12. Sponsoring Agency Name and Address United States Department of Transportation Federal Highway Administration 1200 New Jersey Ave., SE Washington, DC 20590		**13. Type of Report and Period Covered** **14. Sponsoring Agency Code** HOTM	
15. Supplementary Notes GTM: John Halkias, Federal Highway Administration			
16. Abstract The Highway Capacity Manual Reference Guide (HCMRG) is intended to provide simple explanations and applicable guidance for use in typical highway capacity analysis tasks. This Guide covers the analysis and review of all methodological chapters, some of which are quite complex and contain many computations that can be misleading or misunderstood – which is where the guidance becomes most beneficial. The technical approach taken within the HCMRG was to intentionally not repeat the HCM procedures themselves, but to provide key insights to critical parameters and their effects on results that would be especially useful in reviews. The Guide is organized by HCM 2010 chapters with specific references included with each topic. Preceding the topics, a list of changes implemented in the HCM 2010 is also provided for each chapter. This list has been expanded to include modifications in the HCM 6th Edition, including modified references throughout.			
17. Key Words Highway Capacity Manual (HCM), Signalized Intersections, Urban Streets, Interchange Ramp Terminals, Alternative Intersections and Interchanges, Roundabouts, Freeway Segments, Freeway Facilities		**18. Distribution Statement** No restrictions.	
19. Security Classif. (of this report) Unclassified	**20. Security Classif. (of this page)** Unclassified	**21. No. of Pages** 38	**22. Price** N/A

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized.

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Foreword

Welcome to the Federal Highway Administration (FHWA) Highway Capacity Manual (HCM 2010) Reference Guide (HCMRG). This document is intended to provide simple explanations and applicable guidance for use in typical highway capacity analysis tasks. This Guide covers the analysis and review of all methodological chapters, some of which are quite complex and contain many computations that can be misleading or misunderstood – which is where the guidance becomes most beneficial.

The technical approach taken within the HCMRG was intentionally not to repeat the HCM procedures themselves, but to provide key insights to critical parameters and their effects on results that would be especially useful in reviews. The Guide is organized by HCM 2010 chapters with specific references included with each topic. Preceding the topics, a list of changes implemented in the HCM 2010 is also provided for each chapter. This list has been expanded to include modifications in the HCM 6th Edition, including modified references throughout.

Table of Contents

List of Acronyms

AWSC	All-Way Stop Control
CAF	Capacity Adjustment Factor
CFI	Continuous Flow Intersection
d_1	Uniform delay
DLT	Displaced Left Turn
DDI	Diverging Diamond Interchange
DCD	Double Crossover Diamond
L_{EQ}	Equilibrium Distance
g/C	Green-to-cycle Ratio
HCM	Highway Capacity Manual
HCMRG	Highway Capacity Manual Reference Guide
IQA	Incremental Queue Accumulation
d_3	Initial queue delay
NWL	Minimum weaving maneuver lanes
LOS	Level of Service
MUT	Median U-Turn
PHF	Peak Hour Factor
P_{FM} or P_{FD}	Proportion of (merging or diverging) vehicles in Lanes 1 and 2
RCUT	Restricted Crossover U-Turn
RTOR	Right-Turn on Red
TWSC	Two-Way Stop Control
VR	Volume Ratio
v/c	Volume-to-capacity Ratio
W	Weaving intensity factor
L_{MAX}	Weaving segment length exceeding the maximum

Background

The HCM 2010 is the Fifth Edition of this fundamental reference document and includes significant changes. It is organized into four volumes with three printed and the fourth completely online: Volume One – Concepts; Volume Two – Uninterrupted Flow; Volume Three – Interrupted Flow; and Volume Four – Applications Guide.

Over five million dollars in research is included in the updated material contained in the HCM 2010. While every methodological chapter was updated to some extent, the most significant changes were made to Freeway Weaving Segments, Roundabouts, Signalized Intersections and Urban Streets. It is the most complex version yet with several models requiring computational engines to develop and document.

HCM 2010 is the first edition to

- provide an integrated multimodal approach to the analysis of automobile drivers, transit passengers, bicyclists, and pedestrians;
- address both the application of microsimulation analysis to overcome HCM 2010 limitations and the evaluation of those results;
- discuss active traffic management in relation to both demand and capacity;
- include example applications of its procedures implemented in software code to assist in understanding details of the methodologies;
- provide generalized service volume tables to assist planners;
- have an online volume with methodological appendices, case studies, a technical reference library and a mechanism for user interaction with formal HCM 2010 updates.

This list has been expanded to include modifications in the HCM 6th Edition, including modified HCM references throughout. Major additions include procedures for:

- Travel Time Reliability and Active Transportation Demand Management on Urban Streets and Freeway Facilities;
- Managed Lanes, Truck Procedures, Capacity and Speed Adjustments for Weather and Incidents on Freeway Segments and Facilities;
- New Capacity Models and Corridor Analysis for Roundabouts;
- Lane Blockage and Sustained Spillback for Urban Streets; and
- Alternative Intersections and Interchanges that include Diverging Diamond Interchanges (DDI); Displaced Left Turn (DLT) Intersections; Restricted Crossing U-Turn (RCUT) Intersections; and Median U-Turn (MUT) Intersections.

Introduction

Significant discussions with end users on how to apply and review analyses that implement the updated HCM 2010 methodologies provide evidence that there is a need to better understand the inner workings of many input parameters as well as the meaning of intermediate and final results. Gaining insights into which factors are critical to each portion of the results and recognizing misunderstood or misused parameters, as well as the flags that can alert reviewers to incomplete or incorrect analysis techniques, are imperative to providing the resources necessary to analyze or review these analyses adequately.

Before proceeding through the chapter-by-chapter changes and topics, it is valuable to review a few notes that cross-cut all methodologies. Note that some sections begin with this ✔ symbol that indicates a critical point to scrutinize in conducting or reviewing an analysis.

Calibration: A very important and often overlooked step in most analyses is to adjust key base values to match local conditions. These include: 1) base saturation flow rate for signalized intersections, which are also part of urban streets and interchange ramp terminals; 2) base critical and follow-up headway times for two-way stop-controlled intersections; and 3) capacity adjustment factor for freeway facilities, which includes basic freeway segments.

Failure to calibrate for local conditions would normally mean these parameters that are primary in computing results for the referenced procedures would be incorrect and therefore jeopardize realistic results for any of these analyses. Every effort should be made to generate these values of obtain them from the local jurisdiction. Using the HCM 2010 default values is almost always incorrect.

✔ The procedures for calibrating these parameters are described in this HCMRG within each of the respective chapters as priority topics: Signalized Intersections; Two-Way Stop Control; and Freeway Facilities.

Peak Hour Factor: One general parameter that affects every methodology is the peak hour factor (PHF). Adjusted flow rate is used to compute the volume-to-capacity ratio used for calculating delay for interrupted flow procedures and density for uninterrupted flow methods – both used to determine level of service. Care must be taken to get the PHF correct by collecting demand information 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. (Default 0.92 / Typical 0.90 / Range 0.25-1.00)

✔ Even minor adjustments to the PHF can have significant effects on the adjusted flow rate, which in turn changes critical components to service measure computations. *For example, a PHF of 0.90 generates a flow rate of 1111 from a coded volume of 1000, but a PHF of .70 generates a flow rate of 1429 from a coded volume of 1000 – and the flow rate is used in all calculations.*

Software: From the reviewer perspective, two suggestions that apply to all methodologies analyzed using software would be to ensure the software is faithful to the HCM in its implementation of the procedures.

Computer files (not just paper report submittals) should be required to be able to verify results and to expand to problem-solving activities by the reviewer if modifications are necessary. *For example, a few clicks to optimize signal timing using the submitted data file could make a big difference in the results before designs are actually implemented.*

Simulation: When the HCM limitations are encountered and simulation is used to overcome them, the guidance in the HCM for each procedure should be followed to generate results in the most HCM-compliant way. Even with this, users must understand that simulation tools normally have non-HCM algorithms to compute delay, density, and other HCM service and performance measures. Alternate tools that compute HCM-compatible service measures must be verified to ensure results are appropriate for reporting level of service (LOS).

Signalized Intersections
HCM Chapters 19 and 31

HCM 2010 Changes

- Actuated controller settings data are now required, including minimum green, maximum green, passage time, force off, and dual-ring (NEMA) phasing definitions for modeling actuated control.

- A new phase duration model has been implemented to estimate average green times for all actuated phases in fully-, semi-, and coordinated-actuated operations to generate the basis for effective green times.

- An Incremental Queue Accumulation (IQA) model has been incorporated to expand the definition of uniform delay (d_1) including accounting for the proportion of vehicles arriving on green directly.

- Work Zones are modeled by adjusting the saturation flow rate as functions of closed lanes and/or reduced approach width.

HCM 6th Edition

- Unsignalized Movement Delay can now be incorporated into the approach and intersection delay computations.

- Heavy Vehicle and Grade Factor combines the current saturation flow rate adjustments for heavy vehicles and grade to better capture the synergy of the two parameters.

- Critical volume-to-capacity ratio (X_c) is restored with additional guidance for protected-permitted left-turn movements.

- Auxiliary Thru Lane (ATL) volumes are predicted for modeling its effect on Continuous Thru Lane (CTL) capacity and delay using NCHRP Report 707.

- Planning Method is for a sketch-level analysis to evaluate geometry and phasing alternatives, with Part I calculating critical flow ratios for capacity comparisons and Part II extending to v/c ratios, delay and level of service estimates.

Signalized Intersections Topics	✔
Base Saturation Flow Rate Calibration	
Signal Operations	
Phase Duration	
Queue Storage Ratio	
Arrival Demand	
Multiple-Period Analysis	
Unsignalized Movements	
Planning Method	
Intersection Delay	
Saturation Flow Rate Adjustments	
Lane Groups	
Auxiliary Thru Lanes	
Signal Phasing	
Effective Green Time	
Critical v/c Ratio	
Back of Queue	
Average Delay	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

<u>Base Saturation Flow Rate Calibration</u>: Default values of 1900 veh/h/ln and 1750 veh/h/ln are provided for populations of over and under 250,000, respectively. (Default 1900 / Typical 1750 / Range 1300-2300).

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 critical for accurate results within this procedure, since these rates can vary dramatically by jurisdiction. *For example, larger cities typically have base rates well over 2000, while smaller towns can be well under 1600 – significantly changing the basis for capacity, ultimately used to compute delay and level of service.*

[HCM Pages 31-106 and 31-109]

<u>Signal Operations</u>: Traffic signals can be operated in fully actuated, coordinated-actuated, semi-actuated or pretimed 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 pretimed mode to facilitate coordination by maintaining a constant cycle length.

Pretimed – All phases have a fixed duration with no detection used, maintaining a constant cycle length.

Semi-Actuated – Only minor movements have detection with major movements operating as non-actuated and having a fixed duration.

Coordinated-Actuated – Similar to semi-actuated, the major movement is non-actuated, but duration varies to compensate for the minor movement variation for a constant cycle length.

Fully Actuated – All phases operate with detection and each varies with demand.

✔ 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 Pages 31-1 and 31-2]

<u>Phase Duration</u>: This 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 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: The phase duration results are average times over the fifteen-minute analysis period and 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. *For example, reverting to the HCM 2000 analysis by fixing the phase durations to the coded timing parameters completely overrides the computation of average phase times for actuated movements. This significantly affects the computation of effective green time that is used in capacity calculations, ultimately used for delay and level of service.*

[HCM Pages 19-12 thru 19-14 and 31-2 thru 31-22]

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.

Note: One option for approximating this effect would be to use the lane utilization adjustment to saturation flow rate to mimic the reduction in capacity that spillover causes.

✔ The HCM 2010 procedures *do not model* the effects of queue spillover ~~and spillback~~ 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 level of service. Simulation is recommended to model this situation. *For example, any movement with a queue storage ratio greater than 1.0 defines those results (and likely affects adjacent lanes and/or the entire approach) and places all results into question – delay and level of service in these situations are not defendable and should not be accepted.*

[HCM Pages 31-63 and 31-77]

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.

Note: Another method is to quantify unmet demand at the beginning of the red phase for each movement for each cycle for use in determining 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 the following example:

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 arrival rate (not departure flow) was measured.*

[HCM Page 19-15]

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 level of service results. Otherwise, the initial queue (d_3) delay that builds and dissipates over the peak period is not considered in these computations. This analysis must begin and end with undersaturated periods to capture the complete oversaturated process.

Note: Below, single- versus multiple-period analysis comparisons illustrate the effects of unmet demand on the delay and queue results that can be orders of magnitude different:

Delay

v/c Ratio	Single Period	Maximum Multiple Period	Difference	% Difference
0.24	35.10	35.10	0.00	0%
0.42	38.30	38.30	0.00	0%
0.56	41.70	41.80	0.10	0%
0.79	38.30	38.30	0.00	0%
0.96	77.40	78.60	1.20	2%
1.16	138.60	284.30	145.70	105%
1.35	198.70	452.40	253.70	128%
1.54	302.60	1,427.00	1,124.40	372%
1.74	391.20	1,287.00	895.80	229%
1.87	448.20	2,294.00	1,845.80	412%
1.99	498.90	2,452.00	1,953.10	391%

Queue

v/c Ratio	Single Period	Maximum Multiple Period	Difference	% Difference
0.24	5.20	5.20	0.00	0%
0.42	8.70	8.70	0.00	0%
0.61	12.60	12.60	0.00	0%
0.79	17.20	17.20	0.00	0%
0.97	24.50	24.50	0.00	0%
1.16	63.10	101.00	37.90	60%
1.35	38.40	86.40	48.00	125%
1.54	49.90	166.00	116.10	233%
1.75	63.20	270.00	206.80	327%
1.91	73.80	342.00	268.20	363%
2.07	84.70	365.00	280.30	331%

✔ For any analysis where demand may approach capacity, a *multiple-period analysis is mandatory* to ensure the initial queue (d_3) delay is computed to quantify the effects of oversaturation on overall delay and queues. *For example, the above table illustrates that for a volume-to-capacity ratio of 1.5 (not uncommon), the delay for the appropriate multiple-period analysis can be 372% of the inappropriate single-period analysis and the queue for the appropriate multiple-period analysis can be 233% 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 extremely under-designed.*

[HCM Pages 19-19, 19-55, 19-57 / Equation 19-44]

<u>Unsignalized Movements</u>: Delay of unsignalized movements should be included in the approach and intersection aggregate delay and level of service calculations, except for special cases which are properly annotated in the results.

Zero Delay – Delay of many such unsignalized movements, such as free-flow right turns, have zero delay and are easily included in the analysis.

Non-Zero Delay – Delay for other unsignalized movements will need to be estimated by means external to the HCM. External means of estimation might include such things as direct field measurement, observation of similar conditions, special application of other models from the HCM, and simulation.

✔ Approach and Intersection Averages – When the delay of unsignalized movements is included in the approach and intersection averages, whether zero or non-zero, the aggregate delay results must be annotated with a footnote that indicates this unsignalized delay inclusion. Analysts have the option to designate the delay of unsignalized movements that is not included in the aggregate totals, clearly represented by a footnote that indicates unsignalized delay exclusion.

[HCM Page 19-32]

Planning Method: This method is added to provide a planning-level analysis that consists of two parts.

Part I provides an estimate of the intersection's critical flow ratio under a given demand level to assess whether the intersection is likely to operate under, near, or over capacity, predicated using critical movement analysis techniques from Transportation Research Circular 212, and requires only two inputs: turning movement volumes and lane geometry, with other inputs allowed.

Part II extends the results from Part I using more information about phasing to compute effective green times and estimate the effects of coordination.to then produce estimates of volume-to-capacity ratio, delay, and level of service at a lane group level which can then be aggregated to the approach and intersection levels.

[HCM Chapter 31 Section 5]

Intersection Delay: Volume-weighted averaging among lane groups for approach delay, and among approaches for intersection delay, can generate misleading delay and level-of-service 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. Also, adding traffic (like for a traffic impact analysis) to approaches with the least delay (previously undeveloped) could result in a reduction in the intersection delay – also quite misleading.

[HCM Equations 19-28 and 19-29]

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 to really become the engine to the signalized intersection analysis methodology. 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.

[HCM Equation 19-8]

Note: Field data collection should include much more than just turning movement demands in order to adequately model signalized intersections. For the reasons described below, information on heavy vehicles, parking maneuvers, buses stopping, lane utilization, pedestrians, and bicycles must be collected at the same time as the traffic counts are made.

Lane Width – Widths from 10.0 to 12.0 feet receive no adjustment, which is a change from the HCM 2000, where there were reductions at 11 feet and below and increases for 13 feet and above. This difference could affect any comparisons with older analyses. (Default 12.0 / Typical 12.0 / Range 8.0-16.0)

[HCM Exhibit 19-20]

Heavy Vehicles and Grade – Replacing the Heavy Vehicle and Grade adjustments with a combined factor, this adjustment that 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. (Default 3 and 0 / Typical 3 and 0 / Range 0-50% and -4%-+10%)

[HCM Equations 19-9 and 19-10]

Parking – On-street parking is considered if it is within 250 feet of the stop line. As the number of parking maneuvers increases, saturation flow rate decreases even further. The default maneuver time is 18 seconds (based on parallel parking) and should be decreased substantially for angle parking. (Default 0 / Typical 8-32 / Range 0-180)

Note: Even with zero maneuvers per hour, 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 side or far side, are considered stopping buses. If they do not stop, they are modeled as heavy vehicles, but never both. The default bus stop time is 14.4 seconds 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. (Default 0 / Typical 2-12 / Range 0-250)

[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 Central Business District (CBD) of a city, whether or not the intersection is actually in the official CBD. A college campus is a good example of an area that could have these characteristics without being near the center of a city. (Default 1.0 / Typical 1.0 / Range 0.9 or 1.0)

[HCM Page 19-47]

Lane Utilization – The HCM assumes that the lane distribution in a multilane lane group is not equal 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 lane group is used to determine this adjustment – even an estimate can be much better than the default values when these situations exist. (Default Exhibit 18-30 / Typical Exhibit 18-30 / Range 0.333-1.000)

Note: As the lane group demand approaches capacity, the lanes become closer to being equally utilized with little or no reduction in saturation flow rate.

[HCM Equation 19-7 and Exhibit 19-15]

Right Turns – Right-turning vehicles have higher average headway times in order to navigate the tight radius of the turning movement. This adjustment uses a default value for passenger-car equivalents equal to 1.18 that results in a factor of 0.847, reducing the saturation flow rate about 15 percent. The default assumes a turn radius of about 35 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 make a significant difference in the rate. The passenger-car equivalent can be computed from this table by dividing the adjustment into 1.00, and can then be used in the adjustment equation.

Note: While modeling free right turns is not covered explicitly in the HCM 2010, there is a technique that can be used. Adjust the passenger-car equivalent for the appropriate radius (see above) for that portion of the demand moving during the green in the right-turn lane. Compute the capacity separately using the TWSC procedure to estimate that portion of the demand moving on the red to be modeled as Right-Turn On Red (RTOR). This assumes a Stop or Yield condition as the right turn enters the minor street. If there is a separate receiving lane, the lane and demand should be eliminated from the analysis as suggested by the HCM 2010.

[HCM Equation 19-13]

Left Turns – Left-turning vehicles have higher average headway times in order to navigate the radius of the turning movement. This adjustment uses a default value for passenger-car equivalents equal to 1.05 that results in a factor of 0.952, reducing the saturation flow rate about 5 percent. The default assumes a turn radius of about 115 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 make a significant difference in the rate. The passenger-car equivalent can be computed from this table by dividing the adjustment into 1.00, and can then be used in the adjustment equation.

Note: 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 1.23 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 require an adjustment to the saturation flow to account for the increased headway times. Pedestrian counts for all approaches must be included in the analysis if this conflict is considered significant. (Default 0 / Range 0-5000)

[HCM Chapter 31 Section 2]

Bicycles – Bicycles can conflict with right-turning vehicles, which require an adjustment to 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. (Default 0 / Range 0-1900)

Note: If a bicycle lane is to the left of an exclusive right-turn lane where the conflict occurs significantly back from the stop line, eliminating the bicycle count should be considered.

[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 the work zone activity.

[HCM Equations 31-89 thru 31-91]

Lane Groups: Becoming familiar with all lane group possibilities as implemented can be important to understand 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.

[HCM Exhibit 19-19]

Auxiliary Through Lanes: Modeling the effects of Auxiliary Through Lanes (ATLs) by predicting the volume using the ATL by equalizing v/s ratios for pretimed operation. For actuated operation, the non-ATL case is modeled to obtain phase durations for use in computing ATL volumes that are subtracted from the non-ATL case for rerunning until ATL volume predictions converge to within 10 veh/h.

ATL Volume Prediction – NCHRP Report 707 defines the computation of the ATL volume as a function of the thru volume-to-capacity ratio. Equations are provided for one or two Continuous Thru Lanes (CTL) in Equations 3-2 & 3-3 and 3-4 & 3-5, respectively. There are also definitions for upper-bound values in Equations 3-6 (one CTL), 3-7 (two CTLs) and 3-8 (shared thru-right ATL).

[HCM Page 19-31]

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. Major-street thru phases are assigned numbers 2 and 6 with left-turn phases being assigned 1 and 5, all 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 thru phases) or lagging (after the adjacent thru phases). Lead-lag phasing occurs when the left-turn phases for one direction leads and lags for the other direction in protected-only mode.

Note: Lead-lag phasing in combination with protected-permitted phasing must be designated as 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.

[HCM Exhibit 19-4]

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 two seconds) 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. (Default 2.0 / Typical 2.0)

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 – after 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 – 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, where vehicles enter the intersection on yellow – a default value of two seconds is suggested.

Note: Extension of Effective Green time increases with driver aggression created during congestion and/or long cycle lengths, and sometimes long clearance intervals that teach drivers that they have more time than they assumed.

[HCM Equation 19-3 / Exhibit 19-7]

Critical v/c Ratio: Critical volume-to-capacity ratio (X_c) is restored with additional guidance for protected-permitted left-turn movements. This generates a result that was available and used by practitioners prior to the HCM 2010 release.

[HCM Pages 19-57 thru 19-62]

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.

[HCM Chapter 31 Section 4]

Average Delay: Delay reported in seconds per vehicle does not account for the number of vehicles being delayed. In other words, an average delay of 60 sec/veh for a movement with a demand of

1000 veh/h is considered equal to that same average delay of 60 sec/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 3600) can make for much better comparisons, especially when prioritizing improvement projects with an eye to the overall benefit to the public.

Urban Streets

HCM 2010 Chapters 16, 17, 18, 29 and 30

HCM 2010 Changes

- A flow profile model has been implemented to estimate platoon dispersion combined with access point flow rates to generate the proportion of vehicles arriving on green.
- Access points between signalized intersections are now modeled to estimate their effects on delay to through vehicles, platoon decay, and overall travel speed along the segment.
- A procedure for estimating free-flow speed has been incorporated along with a running time method to be used in generating travel speeds along the arterial.
- Level of service is now determined by travel speed of thru vehicles expressed as a percentage of base free-flow speed without arterial classes.

HCM 6th Edition

- Level of Service is now based on average travel speed as a function of base free-flow speed (replacing percent base free-flow speed).
- Lane Blockage is now modeled for a downstream lane closure by adjusting the saturation flow rates of the affected movements discharged to the downstream segment.
- Sustained Spillback from the downstream intersection generates saturation flow adjustment for the movements entering the intersection.
- Right Turns on Red (RTOR) are now included in the flow balancing process to ensure entering and exiting traffic volumes are equal between signalized intersections bounding a segment.
- Free-Flow Speed Calibration is now part of the Base Free-Flow Speed computation to allow for adjustments to local conditions.
- Free-Flow Speed Parking Activity has been added to the adjustments for determining Base Free-Flow Speed.
- Travel Time Reliability procedures have been defined for urban street facilities to model the distribution of factors, such as weather, incidents, work zones, etc. over an extended time.
- Active Transportation Demand Management (ATDM) configurations and control adaptations can be applied to evaluate urban street facilities within the travel time reliability framework.
- Roundabout Corridors are now incorporated into this methodology to evaluate urban street segments bounded by roundabouts – covered under Roundabouts in this Guide.

Urban Streets Topics	✔
Level of Service	
Flow Profile	
Lane Blockage	
Sustained Spillback	
Access Points	
Upstream Filtering	
Flow Balancing	
RTOR Balancing	
Base Free-Flow Speed	
Arrival Type	
Optimizing Timing	
Roundabout Corridors	
Travel Time Reliability	
Active Transportation Demand Modeling	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

Level of Service: Average travel speed of thru vehicles now determines level of service, instead of average travel speed as a percent of base free-flow speed. The threshold for LOS A changed from 85 percent base free-flow speed to the equivalent of 80 percent base free-flow speed. Other level of service results could change for those on the boundaries because of new unit 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 using the arrival type is *almost never 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 / Chapter 30 Section 3]

Note: Side-street approaches are very rarely coordinated with the major-street signals, so these arrival type 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 capacity of the downstream segment as influenced by the midsegment lane restriction is computed and the two values are compared. If the movement capacity exceeds the downstream segment capacity, then 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 the 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: 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 the speeds between signals and arrival rates at signals enough to change the running time and approach delays, creating differences in level-of-service values.

[HCM Equation 18-13 / Chapter 30 Section 4]

✔ For segments that include access points between signals with flow rates that can affect the travel speed and platoon integrity, data must be collected to analyze them within the procedure since the effects can be quite significant. *For example, several side streets or driveways (or fewer with higher volumes) can reduce travel speed and/or the proportion of vehicles arriving on green enough to change the segment level of service.*

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

✔ It is *almost never 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 by 10-20 seconds for movements with volume-to-capacity ratios over 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 not normally balanced. For the evaluation to work properly, 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 Pages 18-25 and 18-26 / Exhibits 18-9 and 18-10]

RTOR Balancing: Right-Turn-On-Red (RTOR) volumes have now been incorporated in the flow profile process to account for these movements as they affect platoons and proportion arriving on green. This is necessary to overcome the elimination of these flows in the Signalized Intersection procedure.

[HCM Pages 30-3 thru 30-5]

Base Free-Flow Speed: 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 now permits the adjustment of base free-flow speed if field data are available and can be applied for overall local conditions or specific street types. A procedure for measuring free-flow speed in the field is available in HCM Chapter 30.

[HCM Pages 18-28 & 18-29 and 30-41 & 30-42]

Parking Activity – This adjustment factor for on-street parking has been added to the base free-flow speed 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]

Free-Flow Speed – Computed as an adjustment to base free-flow speed, this value is no longer allowed to be less 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 with an urban street, except for boundary or side-street approaches where information about the upstream signal is not within the scope of the analysis.

[HCM Pages 18-32 thru 18-34]

<u>Optimizing Timing</u>: 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, there is alternative tools guidance.

Note: This can be accomplished using the HCM procedures with a generic algorithm process on several objective functions to minimize delay, stops, or travel time or to maximize speed or percent base free-flow speed for the best level of service.

[HCM Chapter 29 Section 4]

<u>**Roundabout Corridors**</u>: 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]

Base Free-Flow Speed – This parameter is computed exactly the same for segments bounded by roundabouts and signalized intersections.

[HCM Equation 30-72 / 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, the average radius of the thru circulating path, the circulating speed and the subsegement lengths.

[HCM Exhibits 30-40 thru 30-42]

Free-Flow Speed – The free-flow speeds are computed for Subsegment 1 and Subsegement 2 as functions of the influence areas, and may be lower than the speed limit (unlike segments bounded by signalized intersections). The free-flow speeds for segments without roundabout influence are computed exactly the same as for segments bounded by signalized intersections for comparison with the subsegment values where the minimum of the three is used to computer running time.

[HCM Equations 30-73 thru 30-86]

Running Time – This equation is modified for yield control at roundabouts with a start-up lost time of 2.5 (not 2) seconds and limiting the first term to the volume-to-capacity 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 thru flow rate.

[HCM Chapter 22 / HCM Equations 30-88 and 22-17]

Geometric Delay – The segment geometric delay is computed for each subsegment as a function of free-flow and circulating speeds with the inscribed circle diameter.

[HCM Equation 30-89 and 30-90]

Thru Delay – Delay for the thru 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 level of service exactly as for segments bounded by signalized intersections.

[HCM Equation 18-15 / Exhibit 18-1]

<u>Travel Time Reliability</u>: This methodology provides for the generation of a distribution of trip travel time over an extended period of time 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 distribution generation of scenarios.

[HCM Page 17-12]

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

[HCM Page 17-15 thru 17-17 and Page 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 Pages 17-17 and 17-23]

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

[HCM Page 17-18 thru 17-22 and Pages 17-23 thru 17-24]

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

[HCM Pages 17-22 and 17-24]

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

[HCM Pages 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 Page 17-26 / HCM Chapter 29 Section 2]

Travel Time Index – 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 Pages 17-9 thru 17-10 and 17-28 thru 17-30]

Planning Time Index – PTI is defined as the ratio of the 95th percentile highest travel time to the travel time at the base free-flow speed.

[HCM Pages 17-10 and 17-30]

Active Transportation Demand Management: 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 and signal control (like adaptive signal timing and priority treatments) and/or modifications of geometric configurations (like reversible lanes and dynamic lane or turn-lane assignments).

[HCM Chapter 17 Section 4]

Ramps Terminals and Alternative Intersections

HCM 2010 Chapters 23 and 34

HCM 2010 Changes

- The procedures adopted in the TRB Circular for the HCM 2000 were modified to be consistent with Signalized Intersections and Urban Streets.

HCM 6th Edition

- The ramp terminals procedures were modified to be consistent with other intersection and interchange types within this chapter to use Experienced Travel Time (ETT) to include Extra Distance Travel Time (EDTT) where applicable to determine level of service.

- Median U-Turn (MUT) intersections are now modeled for both signalized and stop-controlled crossovers at the supplemental intersections.

- Restricted Crossing U-Turn (RCUT) intersections are now modeled for signalized, stop-controlled and merging crossovers at the supplemental intersections.

- Displaced Left Turn (DLT) intersections are now modeled for signalized crossovers at the supplemental intersections for both partial and full designs.

- Diverging Diamond Interchanges (DDI) are now modeled for both signalized and yield-controlled off-ramp designs.

Interchange Ramp Terminals Topics	✔
Level of Service	
Experienced Travel Time (ETT)	
Signalized Intersections	
Lane Utilization	
Saturation Flow	
Downstream Queue	
Demand Starvation	
Effective Green Time	
Alternative Intersections/Interchanges	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

Level of Service: Results are computed for each origin-destination (O-D) pair by computing the Experienced Travel Time (ETT) for each pair for comparison with the level-of-service thresholds as defined. A volume-to-capacity ratio or a queue-storage ratio greater than 1.0 for either lane group results in LOS F for the O-D pair.

Note: Lane groups and intersections are not considered for level of service except to check volume-to-capacity and queue-storage ratios.

[HCM Exhibits 23-10 and 23-12]

✔ A comparison of movement delays from each intersection to the sums used for level of service can reveal interactive issues. *For example, the series of O-D delays could generate acceptable levels of service when compared to the Interchange Ramp Terminals thresholds, but be less acceptable when scrutinized by individual movements compared to the Signalized Intersection thresholds.*

Experienced Travel Time (ETT): For these distributed intersection, 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, additional geometric information must be provided to be able to compute these results correctly, including the extra distance travelled along the ramp and the design speed of the ramp.

Note: The EDTT value can be negative for right turns because of the destination heading away from the freeway centerline creating a net savings in distance travelled.

[HCM Equations 23-1 thru 23-10 / Exhibits 23-6 thru 23-9 and 23-11 thru 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 level of service. Several factors affecting saturation flow rates and effective green times are modified for signalized intersections as part of interchanges.

[HCM Pages 23-5 and 23-6]

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 thru 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 further enhance the saturation flow rate adjustment can be critical to results.

Note: Turn radius equivalencies can be useful in Chapter 19 analyses for skewed intersections or other non-standard designs.

[HCM Equations 23-15, 23-19 thru 23-23 / 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 / Equations 23-29 thru 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.

[HCM Exhibit 23-28 / 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 thru 23-28]

Alternative Intersections/Interchanges:

"Distributed intersections" consist of groups of two or more intersections that, by virtue of close spacing and displaced/distributed traffic movements, are operationally interdependent and are thus best analyzed as a single unit.

[HCM Page 23-1]

- 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.

[HCM Page 23-5]

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

[HCM Page 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 turns are not rerouted.

[HCM Page 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 Page 23-5]

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 origin-destination values.

Two-Way Stop Control (TWSC)

HCM 2010 Chapters 20 and 32

HCM 2010 Changes

- The procedures were expanded to analyze intersections with up to three through lanes on the major street, including the effects of U-turns on conflicting flow and gap acceptance.

- The effects of upstream signals on conflicting flow and capacity now use the proportion time blocked from the Urban Streets methodology to implement this modification.

- The estimation of delay to Rank 1 vehicles now includes major-street approaches with left-turn lanes where the queue exceeds the storage.

Two-Way Stop Control Topics	✓
Calibration	
Two-Stage Gap Acceptance	
Upstream Signals	
Peak Hour Factor	
Rank 1 Delay	
Queuing	
Level of Service	
Average Delay	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

<u>Calibration</u>: 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 have significant effects the gap acceptance by drivers.

Note: The process for collecting critical and follow-up headway data in the field is quite difficult, but measuring delay is relatively simple (following the procedure outlined on HCM Pages 31-99 thru 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 larger demands. *For example, when traffic levels are high (like during peak periods) or drivers are aggressive (like in larger cities) the default values can yield much higher delays and queues than really exist because drivers will actually be accepting much shorter gaps.*

[HCM Pages 20-18 and 20-19 / HCM Exhibits 20-12 and 20-13]

Two-Stage Gap Acceptance: This application differs depending on median type with both left-turning traffic and through traffic modeled in two stages for medians, but with only left-turning traffic modeled in two stages for TWLTL.

✔ The number of storage spaces in the median should be defined as it functions. Data should be provided to support using more than one vehicle, since this can have a significant effect on results. *For example, changing one vehicle to two vehicles can lower delay significantly and should only be allowed when justified by field data.*

[HCM Exhibits 20-10 and 20-11]

Upstream Signals: The effects of upstream signals on the conflicting flow rates are modeled using the proportion time blocked results from an Urban Streets analysis.

[HCM Equations 20-33, 20-34 and 20-35]

✔ The Urban Streets data set should be reviewed to confirm the parameters used to compute the proportion time blocked values. *For example, approximations should not be accepted since this is a very complex model and can affect results significantly.*

Peak Hour Factor (PHF): Expanded guidance has been added for using multiple fifteen-minute period analyses in place of one hourly analysis when approaches have different peaking characteristics. The use of one PHF for the intersection, rather than by approach or movement, is reiterated as well.

[HCM Page 20-12]

Rank 1 Delay: Delay to major-street through and right-turning vehicles 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.

[HCM Exhibit 20-15 / Equations 20-43 and 20-44]

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 Pages 20-32 and 20-33]

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.

[HCM Exhibits 19-8, 20-2, 21-8 and 22-8]

Average Delay: Average delay (in seconds per vehicle) does not account for the number of vehicles being delayed. Vehicle hours of delay (average delay times vehicles per hour divided by 3600) can make for better comparisons, especially when prioritizing projects.

All-Way Stop Control (AWSC)

HCM 2010 Chapters 21 and 32

HCM 2010 Changes

- The procedure was expanded to model intersections with up to three lanes on each approach, increasing the potential number of degree-of-conflict cases to 512.
- The computation of 95th-Percentile Queue (based on the TWSC relationship modified to use departure headway) is now included.

All-Way Stop Control Topics	✔
Lane Utilization	
Lane Configuration	
Queuing	
Level of Service	
Average Delay	

Analysis Topics

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.

[HCM Page 21-13]

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.

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 Page 21-33]

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.

[HCM Exhibits 19-8, 20-2, 21-8 and 22-8]

Average Delay: Average delay (in seconds per vehicle) does not account for the number of vehicles being delayed and that vehicle hours of delay (average delay times vehicles per hour divided by 3600) can make for better comparisons, especially when prioritizing projects.

Roundabouts

HCM 2010 Chapters 22 and 33

HCM 2010 Changes

- The methodology now computes delay and provides level of service for single- and two-lane roundabouts with single- or two-lane approaches.
- A gap acceptance model generated from U.S. data was implemented to compute capacity with critical and follow-up headway values more appropriate for U.S. conditions.
- Right-turn bypass lanes, both yielding and non-yielding, are now analyzed to include their effects on approach and intersection delay and level of service.
- The effects of pedestrians on capacity for single- or two-lane approaches are computed as functions of entering vehicle conflicting flow rate.

HCM 6th Edition

- The capacity equations were revised to reflect additional research and potentially improved driver familiarity in the US, increasing capacity by 20-25%.
- The ability to model roundabout corridors as urban streets segments and facilities is added with geometric delay at roundabouts also introduced.

Roundabouts Topics	✔
Calibration	
Capacity Models	
Lane Utilization	
Bypass Lane Definitions	
Yielding Bypass Lanes	
Non-Yielding Bypass Lanes	
Pedestrians	
Queuing	
Level of Service	
Average Delay	
Data	
Roundabout Corridors	

Analysis Topics

Calibration: Critical headway and follow-up headway times should be calibrated 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.

[HCM Page 33-6]

Capacity Models: All equations for computing capacity have been revised to reflect more recent research and likely the US familiarity that has caused the base values to increase from 1130 to between 1350 and 1420, depending on the lane configuration.

Note: This represents a 20-25% increase in capacity which will significantly decrease delay as compared with all results prior to this change. Most level of service values will change even with exactly the same data.

[HCM Equations 22-1 thru 22-7]

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.

[HCM Exhibits 22-14 and 22-15 / Pages 33-4 thru 33-5]

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 and non-yielding right-turn bypass lanes merge downstream.

[HCM Pages 22-7 thru 22-8 / Exhibit 22-7]

Yielding Bypass Lanes: The exiting flow becomes the conflicting flow for use in computing capacity for yielding right-turn bypass lanes since the merge is at the point of exit.

[HCM Equations 22-6, 22-7 and 22-11]

Non-Yielding Bypass Lanes: Delay for non-yielding right-turn bypass lanes is established at zero by definition and is included in overall intersection average delay. This is in contrast with free right turns at signalized intersections and must be adjusted for comparisons.

[HCM Equation 22-18]

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.

[HCM Pages 22-20 and 22-21 / Exhibits 22-18 thru 22-21]

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 Page 22-24]

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.

[HCM Exhibits 19-8, 20-2, 21-8 and 22-8]

Average Delay: Average delay (in seconds per vehicle) does not account for the number of vehicles being delayed. Vehicle hours of delay (average delay times vehicles per hour divided by 3600) can make for better comparisons, especially when prioritizing projects.

Data: Collecting turning movement count data can be a challenge since entering vehicles must be followed through to their exiting legs to properly consider through movements, left turns and U-turns as they navigate the roundabout.

Roundabout Corridors: 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]

Inscribed Circle – This is defined as the diameter of the largest circle that can be inscribed within the outer edges of the circulatory roadway.

[HCM Exhibit 30-42]

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

[HCM Page 30-77]

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, then included with control delay in travel speed determination.

[HCM Equations 30-89 thru 30-91]

Travel Speed – This is the service measure that determine level of service computed from the combination of running time along the segment and thru delay at the roundabout, converted to speed using the Urban Streets procedure.

[HCM Equation 18-15]

Level of Service – LOS is based on travel speed as a function of base free-flow speed using the Urban Streets thresholds.

[HCM Exhibit 18-1]

Basic Freeway Segments
HCM 2010 Chapters 12 and 26

HCM 2010 Changes

- The Base Free-Flow Speed was changed to 75.4 mi/h with the adjustment for number of lanes eliminated, and interchange density adjustment was replaced with an adjustment for total ramp density.

- The Free-Flow Speed curves were modified, which created new breakpoints and dictated no interpolation among the curves in determining average travel speed.

HCM 6th Edition

- Interpolation among the speed-flow curves for computing average travel speed was restored with adjustments to capacity and speed accommodated.

- A revised Truck Procedures is now implemented to generate passenger-car equivalents (PCE) for heavy vehicle mixes defined by percentages of single-unit trucks (SUT) and tractor trailers (TT).

- Managed Lanes can now be modeled for five types of access with the general purpose lanes, including continuous access, buffers and barriers for one and two managed lanes.

- Capacity Adjustment Factors (CAF) and Speed Adjustment Factors (SAF) are defined for various weather events and incident shoulder and lane closures, including driver population.

Basic Freeway Segments Topics	✔
Speed-Flow Curves	
Free-Flow Speed	
Lateral Clearance	
Truck Procedure	
Managed Lanes	
Adjustments	
Segments	
Travel Speed	
Break Points	
Ramp Density	
Driver Population	
One Direction	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

Speed-Flow Curves: While interpolation among these curves for free-flow speeds other than 75, 70, 65, 60 and 55 mi/h was eliminated in the HCM 2010, it has been restored in the Update. Additionally, if CAF and/or SAF values other than 1.0 are used, the curves are adjusted accordingly with modified break points and capacity values.

[HCM Equation 12-1 and Exhibits 12-5 & 12-6]

Free-Flow Speed: Free-flow speed should be measured when the HCM default (75.4 mi/h) may be inappropriate, and then used directly without any adjustments.

✔ Especially where geometric design has lower standards for lateral clearance or grade (i.e., urban or mountainous situations), field-measured free-flow speed becomes very important since it could be significantly lower than the 75.4 mi/h default (which can be overridden as of the Update). *For example, 75 mi/h was the accepted default for rural freeways in HCM 2000, which assumes few cross-section restrictions.*

[HCM Page 12-27 / Equation 12-2]

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

[HCM Exhibit 12-21]

Truck Procedure: The procedure for incorporating passenger-car equivalents (PCE) has been revised. The heavy-vehicle mixed is now defined as the split between single-unit trucks (SUT) and tractor-trailers (TT), with buses and recreational vehicles considered SUTs.

[HCM Equation 12-10 and Exhibits 12-25 thru 12-28]

Managed Lanes: Managed lanes are modeled for five defined designs, including continuous access, buffer-separated single lane, buffer-separated multiple lanes, barrier-separated single lane, and barrier separated multiple lanes. Capacity values for managed lanes are provided as functions of the flow speed and access design. When the density of the general purpose lanes exceeds 35 pc/mi/ln, friction is assumed and the speed-flow curves are adjusted for the continuous access and buffer single-lane designs.

[HCM Pages 12-12 thru 12-15 / Exhibits 12-11 thru 12-13 / HCM Chapter 12 Section 4]

Adjustments: Capacity and speed adjustment factors are now provided for weather events, and with capacity adjustment factors also provided for incident events. The driver population adjustment is now applied to capacity and no longer to flow rate.

[HCM Equations 12-5 & 12-8]

✔ Where the left-side lateral clearance is less than 6 feet, field-measured free-flow speed may be warranted. *For example, the significant effects of a guardrail within a few feet of vehicles would otherwise be ignored.*

Segments: Segments should be homogenous and broken into multiple analyses if noteworthy operating features (number of lanes, free-flow speed, clearances, grades, etc.) vary significantly.

[HCM Page 12-7]

Travel Speed: Free-flow speed is no longer interpolated, but always 55, 60, 65, 70, or 75 as closest to the computed value and cannot be outside that range when used to determine travel speed.

[HCM Page 12-10 / Equation 12-1]

Break Points: Average travel speed is no longer equivalent to free-flow speed when the flow rate exceeds break points that vary by free-flow speed. These flow-rate break points are 1000, 1200, 1400, 1600, and 1800 pc/h/ln for free-flow speeds of 75, 70, 65, 60, and 55 mi/h, respectively.

[HCM Exhibit 12-6]

Ramp Density: Ramp density is determined by counting the ramps (not interchanges) 3 miles upstream and downstream from the analysis segment midpoint then dividing by six to obtain the ramps per mile.

[HCM Page 12-30]

Driver Population: A factor of 1.0 is advised to represent primarily commuters and familiar drivers in the traffic stream, unless there is sufficient evidence of unfamiliar drivers that would require a lower value. This adjustment is now implemented as part of the Capacity Adjustment Factor (CAF) and the Speed Adjustment Factor (SAF). (Default 1.0 / Range 0.85 thru 1.0 / Typical 1.0)

[HCM Exhibit 26-9]

One Direction: It must be recognized that one direction of travel is modeled in each analysis and that an additional analysis is necessary to model the opposing direction.

Freeway Weaving Segments
HCM 2010 Chapters 13 and 27

HCM 2010 Changes

- Boundaries were redefined for determining the weaving segment length to use the short length separated by delineation.
- Computation of the maximum weaving length was defined as a function of volume ratio (VR) and lanes involved in the weave (N_{WL}).
- Capacity was modified to be controlled by either demand as a function of the lanes involved in the weave (N_{WL}) or a density of 43 pc/mi/ln.
- Computation of lane changing rates was introduced to determine the weaving intensity factor (W) for use in computing the weaving speed.
- LOS F was changed to be controlled by the volume-to-capacity (v/c) ratio only and is no longer defined by density.

Note: An update approved in January 2014 defines densities exceeding 43 pc/mi/ln as LOS F.

HCM 6th Edition

- A revised Truck Procedures is now implemented to generate passenger-car equivalents (PCE) for heavy vehicle mixes defined by percentages of single-unit trucks (SUT) and tractor trailers (TT).
- Managed Lanes can now be modeled for entry and exit from barrier designs, and for cross weaving to and from on and off ramps to and from managed lanes.
- Capacity Adjustment Factors (CAF) and Speed Adjustment Factors (SAF) are defined for various weather events and incident shoulder and lane closures, including driver population.

Freeway Weaving Segments Topics	✔
Maximum Weaving Length	
Managed Lanes	
Adjustments	
Lane Changes	
Weaving Lanes	
Driver Population	
One Direction	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

Maximum Weaving Length: Guidance is provided to model three basic freeway segments instead of weaving segments when the maximum weaving length is exceeded.

✔ The maximum weaving length computation can potentially yield unreasonably high distances and should be given a reality check, especially when longer than 1 mile. *For example, it would be unusual for a freeway section to have the friction associated with weaving for over one mile, but the procedure might generate that as a maximum length.*

[HCM Exhibit 13-11]

Managed Lanes: A capacity reduction factor is computed for the cross weave from an on ramp to the managed lane(s) or from the managed lane(s) to an off ramp. The cross-weaving flow rate is used in conjunction with the distances between the ramp gore and the beginning and end of access to the managed lane(s).

[HCM Exhibit 13-12 and Equation 13-24]

Adjustments: Capacity and speed adjustment factors are now provided for weather events, and with capacity adjustment factors also provided for incident events. The driver population adjustment is now applied to capacity and no longer to flow rate.

[HCM Exhibits 11-20, 11-21, 11-23 & 26-9]

Lane Changes: New rules are defined for determining minimum lane changes required for freeway-to-ramp and ramp-to-freeway movements, including two-sided weaves.

[HCM Pages 13-7 thru 13-9 / Exhibit 13-5]

Weaving Lanes: New rules are defined to determine the number of lanes involved in the weave as those from which a movement can be made with one or zero lane changes. Zero ia always used for two-sided weaves.

[HCM Pages 13-7 thru 13-9 / Exhibit 13-5]

Driver Population: A factor of 1.0 is advised to represent primarily commuters and familiar drivers in the traffic stream, unless there is sufficient evidence of unfamiliar drivers that would require a lower value. This adjustment is now implemented as part of the Capacity Adjustment Factor (CAF) and the Speed Adjustment Factor (SAF). (Default 1.0 / Range 0.85 or 1.0 / Typical 1.0)

[HCM Exhibit 26-9]

One Direction: It must be recognized that one direction of travel is modeled in each analysis and that an additional analysis is necessary to model the opposing direction.

Freeway Merge & Diverge Segments

HCM 2010 Chapters 14 and 28

HCM 2010 Changes

- Procedures have been added to check for unreasonable lane distributions that overload the left or right lane(s) (or both) of the freeway.

- A revision has been made to correct an illogical trend involving on-ramps on eight-lane freeways in which density increases as the length of the acceleration lane increases.

- Capacity values were added for high-speed ramp junctions on multilane highways and C-D roadways that also accommodate free-flow speeds as low as 45 mi/h.

Freeway Merge & Diverge Segments Topics	✔
Acceleration/Deceleration Lanes	
Managed Lanes	
Adjustments	
Aggregated Density	
Adjacent Ramps	
Lane Additions/Drop	
Left-Hand Ramps	
Five-Lane Freeways	
Major Merge/Diverge	
Ramp Meters	
Driver Population	
One Direction	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

Acceleration/Deceleration Lanes: As acceleration and deceleration lengths increase, density decreases, which is expected. However, care should be taken to review density results when the length of an acceleration or deceleration lane is above 1500 feet.

✔ 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. *For example, even negative density values can be computed from reasonable data.*

[HCM Equations 14-2 thru 14-26]

Managed Lanes: In the case of a left-side ramp interacting with a Barrier 1 managed lane, the density and LOS can be computed by doubling the managed lane volume to analyze as if there are two managed lanes.

[HCM Exhibit 14-22]

Adjustments: Capacity and speed adjustment factors are now provided for weather events, and with capacity adjustment factors also provided for incident events. The driver population adjustment is now applied to capacity and no longer to flow rate.

[HCM Exhibits 11-20, 11-21, 11-23 & 26-9]

Aggregated Density: Density can be computed across all lanes (not just lanes one and two as is used for level of service) by dividing the total flow rate by the average speed in all lanes.

[HCM Equation 14-24 and Exhibit 14-15]

Adjacent Ramps: The effects of adjacent upstream and/or downstream adjacent 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.

Note: If both upstream and downstream adjacent ramps exist, the analysis resulting in the highest proportion is used.

[HCM Exhibits 14-8 and 14-9 / Equations 14-6, 14-7, 14-12 and 14-13]

Lane Additions/Drops: Lane additions at merges or lane drops at diverges cannot be modeled in this methodology. In the case of an auxiliary lane between two ramps, the situation can be modeled as a weaving segment if the distance between the ramps is within the maximum length computed in Chapter 13. If the distance is longer than the maximum, or if there is no adjacent ramp, the freeway before and after the ramp is modeled as a basic freeway segment.

[HCM Page 14-24 / HCM Exhibit 10-12]

Left-Hand Ramps: When ramps are on the left side of the freeway, the number of vehicles in the lanes closest to the ramps are determined by computing the volume that would be 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.

[HCM Exhibit 14-18]

Five-Lane Freeways: Given that the relationships to determine the proportion of vehicles in Lanes 1 and 2 extend only to four lanes in each direction, there is a table that estimates the proportion in the fifth lane of a five-lane freeway. This volume is subtracted from the total freeway flow to create a similar four-lane segment for use in the procedure.

Note: While the HCM 2010 does not specifically provide for this, the logic could be extended to six- and even seven-lane freeway segments for reductions down to the four lanes necessary to complete the computations as a reasonable extension of this adjustment.

[HCM Exhibit 14-19]

Major Merge/Diverge: A major diverge analysis requires no software with density estimated using one simple equation, but there are no models for a major merge analysis that must be simulated.

[HCM Exhibits 14-20 and 14-21 / Equation 14-28]

Ramp Meters: Results for metered ramps can be approximated by using the demand as limited by the metered rate. However, since the meter affects the merge dynamic by spacing vehicles, simulation must be used for a more accurate model.

[HCM Page 14-35]

Driver Population: A factor of 1.0 is advised to represent primarily commuters and familiar drivers in the traffic stream, unless there is sufficient evidence of unfamiliar drivers that would require a lower value. This adjustment is now implemented as part of the Capacity Adjustment Factor (CAF) and the Speed Adjustment Factor (SAF). (Default 1.0 / Range 0.85 or 1.0 / Typical 1.0)

[HCM Exhibit 26-9]

One Direction: It must be recognized that one direction of travel is modeled in each analysis and that an additional analysis is necessary to model the opposing direction.

Note: In this chapter, two tables refer to total freeway lanes in both directions, which can be very misleading (even with the footnote).

[HCM Exhibits 14-8 and 14-9]

Freeway Facilities
HCM 2010 Chapters 10, 11 and 25

HCM 2010 Changes

- Changes to procedures for Basic Freeway, Freeway Weaving, and Merge & Diverge Segments were incorporated within the Freeway Facilities methodology.

- Facility-wide level of service based on a segment length and lane number weighted average of density applied to Basic Freeway Segments level-of-service thresholds was added.

Freeway Facilities Topics	✔
Calibration	
Oversaturated Conditions	
Travel Time Reliability	
Active Transportation Demand Management	
Managed Lanes	
Truck Procedure	
Work Zones	
Analysis Length	
Segments	
Weaving Segments	
Overlapping Segments	
Time-Space Domain	
Scale Factor	
Facility Capacity	
Weaving Capacity	
Capacity Adjustment	
Work Zone Analysis	
Truck Procedure	
Bottlenecks	
Limitations	
Level of Service	

Analysis Topics

Priority topics are listed first with additional guidance provided for these most important checks, but all analysis topics can be important for a given analysis.

<u>Calibration</u>: Capacity Speed Demand for Free-Flow Speed Bottleneck Incident Weather

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.

✔ The Capacity Adjustment Factor (CAF) can be computed by comparing measured values with those from the segment procedures to produce the appropriate proportion for (generally) reducing the capacity. The CAF is subsequently used to produce adjusted speed-flow curves for application within the procedures.

[HCM Equation 25-1 / Exhibit 25-2]

Oversaturated Conditions: Modeling multiple segments over multiple time periods (with time periods less than 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 time periods containing no segments operating at LOS F. *For example, when the first segment operates at LOS F with a queue extending upstream and when the last segment operates at LOS F with a downstream bottleneck, these cannot be analyzed if not in the facility; similarly, if the first time period is at LOS F, previous time periods could be failing. Conversely, if the last time period is at LOS F, subsequent periods may be failing.*

[HCM Pages 10-17, 10-34 and 10-35]

Travel Time Reliability: This methodology provides for the generation of a distribution of trip travel time over an extended period of time 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 distribution 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, 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.

Travel Time Index – 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 is defined as the ratio of the 95th percentile highest travel time to the travel time at the base free-flow speed.

[HCM Chapter 11]

Active Transportation Demand Management: 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 and signal control (like adaptive signal timing and priority treatments) and/or modifications of geometric configurations (like reversible lanes and dynamic lane or turn-lane assignments).

[HCM Chapters 11 and 25]

Managed Lanes: Implemented for each segment within the facility as defined by the segment procedures.

[HCM Pages 10-46 thru 10-47]

Truck Procedure: Implemented for each segment within the facility as defined by the segment procedures.

Work Zones: Work zones are modeled to generate Capacity Adjustment Factor (CAF) and Speed Adjustment Factor (SAF) values to model the effects of work zones. Parameters considered in developing these factors include a Lane Closure Severity Index (LCSI) 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 (TRD).

[HCM Exhibit 10-15 and Equations 10-8 thru 10-12]

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

[HCM Page 10-25]

Segments: Segments should be homogenous and broken into multiple analyses if noteworthy operating features (number of lanes, free-flow speed, clearances, grades, etc.) vary significantly.

[HCM Page 10-28]

Weaving Segments: Weaving segment length exceeding the maximum (L_{MAX}) are analyzed as a basic freeway segment (modified from the printed HCM).

[HCM Exhibit 10-12]

Overlapping Segments: The distance between adjacent segments where the 1500-ft ramp influence areas overlap becomes an overlap segment.

[HCM Exhibit 10-11c]

Time-Space Domain: The freeway segments and the time periods to be analyzed must be established in order to model the effects of the interaction among the segments with demand varying over time.

[HCM Pages 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 demand for each segment to balance any discrepancies.

[HCM Pages 10-28 and 10-29 / 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.

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

[HCM Page 10-1]

Weaving Capacity: Since demand is used to compute the capacity of a freeway weaving segment, this value can vary among time periods.

[HCM Page 10-30]

Capacity Adjustment: Modifications to capacity can be applied to model the effects of short-term work zones (incorporating intensity of activity, effects of heavy vehicles and presence of ramps) and long-term construction zones (using lane reduction estimates for capacities from several states). Weather and environmental effects are also discussed with adjustments largely left to the user and with limited data for specific guidance.

Note: Lane-width considerations should be incorporated where appropriate to include the effects on free-flow speed in work or construction zones.

[HCM Pages 10-30 and 10-31]

Driver Population: A factor of 1.0 is advised to represent primarily commuters and familiar drivers in the traffic stream, unless there is sufficient evidence of unfamiliar drivers that would require a lower value. This adjustment is now implemented as part of the Capacity Adjustment Factor (CAF) and the Speed Adjustment Factor (SAF). (Default 1.0 / Range 0.85 or 1.0 / Typical 1.0)

[HCM Exhibit 26-9]

Bottlenecks: The effect of a bottleneck (queuing affects upstream segments and capacity restraints meter downstream segments) on adjacent segments and the facility as a whole is extremely important in understanding the results when queuing and delay are part of freeway system.

[HCM Pages 10-12 thru 10-17 and 10-30]

Limitations: Multiple overlapping bottlenecks, HOV lanes, toll plazas, or off-ramps queuing onto the freeway must be avoided as situations not able to be modeled using this procedure.

[HCM Page 10-19]

Level of Service: Results are interpreted from a matrix of values for multiple segments and time periods to determine the worst situation for an overall level of service.

Note: LOS F can exist for a given segment when the queue from a downstream breakdown extends to that segment.

[HCM Page 10-15]

Multilane Highways

HCM 2010 Chapters 12 and 26

HCM 2010 Changes

- Guidance was updated to be more precise in estimating the Base Free-Flow Speed as a function of the posted speed limit.
- Direction for using Free-Flow Speed curves to obtain average travel speed was modified to dictate no interpolation among the curves in determining average travel speed.

Multilane Highways Topics	✔
Speed-Flow Curves	
Travel Speed	
Free-Flow Speed	
Access Points	
Lateral Clearance	
Break Points	
Truck Procedure	
Segments	
Level of Service	
Driver Population	
One Direction	

Analysis Topics

Speed-Flow Relationship: The speed-flow curves are modified to be more consistent with Basic Freeway Segments to have the threshold between LOS E and F at a density of 45 pc/mi/ln. While interpolation among these curves for free-flow speeds was eliminated in the HCM 2010, it has been restored in the Update. Additionally, the range has been extended to include 65 and 70 mi/h curves.

[HCM Equation 12-1 and Exhibits 12-5 & 12-6]

Travel Speed: Free-flow speed interpolation has been restored and no longer relegated to the closest published speed.

[HCM Equation 12-1 and Exhibit 12-6]

Free-Flow Speed: Base free-flow speed is considered to be 5 mi/h over the speed limit (if 50 mi/h or more) and 7 mi/h over the speed limit (if less than 50 mi/h).

[HCM Exhibit 12-18]

Access Points: Access point density is determined by counting access points on the right side only and including left-side access point only if the highway is one-way.

[HCM Page 12-31]

Lateral Clearance: Lateral clearance is assumed to be 6 feet on the left side for undivided highways and 6 feet on either side if the actual clearance is over 6 feet. (Default 12 / Range 0-12 / Typical 12)

[HCM Page 12-30]

Break Points: The break point where average travel speed is no longer equivalent to free-flow speed is when the flow rate exceeds 1400 pc/h/h.

[HCM Exhibit 12-6]

Truck Procedure: The procedure for incorporating passenger-car equivalents (PCE) has been revised. The heavy-vehicle mixed is now defined as the split between single-unit trucks (SUT) and tractor-trailers (TT), with buses and recreational vehicles considered SUTs.

[HCM Equation 12-10 and Exhibits 12-25 thru 12-28]

Segments: Segments should be homogenous and broken into multiple analyses if noteworthy operating features (number of lanes, free-flow speed, clearances, grades, etc.) vary significantly.

[HCM Page 12-21]

Level of Service: The LOS F thresholds, as well as capacity values, vary by free-flow speed.

[HCM Exhibit 12-15]

Driver Population: A factor of 1.0 is advised to represent primarily commuters and familiar drivers in the traffic stream, unless there is sufficient evidence of unfamiliar drivers that would require a lower value. This adjustment is now implemented as part of the Capacity Adjustment Factor (CAF) and the Speed Adjustment Factor (SAF). (Default 1.0 / Range 0.85 or 1.0 / Typical 1.0)

[HCM Exhibit 26-9]

One Direction: It must be recognized that one direction of travel is modeled in each analysis and that an additional analysis is necessary to model the opposing direction.

Two-Lane Highways

HCM 2010 Chapters 15 and 26

HCM 2010 Changes

- Class III highways were added to accommodate modeling within moderately developed areas with level of service based on Percent of Free-Flow Speed.

- The two-way analysis option was eliminated.

Two-Lane Highways Topics	✔
Highway Class	
Access Points	
Free-Flow Speed	
Passing Lanes	
Facilities	
One Direction	

Analysis Topics

Highway Class: The determination of class is somewhat subjective with text descriptions of commuting, access, and moderately developed roadways.

[HCM Pages 15-3 thru 15-5]

Access Points: Counting access points in both directions is appropriate even though the analysis is for one direction, since an access point on the left will still affect free-flow speed.

[HCM Page 15-16 / Exhibit 15-8]

Free-Flow Speed: Measuring free-flow speed still requires adjustments, since any traffic during the field measurement would affect the results.

[HCM Equation 15-1]

Passing Lanes: Average Travel Speed and Percent Time Spent Following results are modified to adjust for the effects of passing lanes using the four distance regions.

Note: A passing lane is considered to be a climbing lane when the upgrade has more than 200 veh/h with at least 10 percent trucks and a speed reduction of at least a 10 mi/h.

[HCM Pages 15-36 and 15-37 / Exhibit 15-29]

Facilities: Segments that have different operating features (free-flow speed, clearances, grades, etc.) can be combined into facilities.

[HCM Pages 15-28 and 15-29]

One Direction: It must be recognized that one direction of travel is modeled in each analysis and that an additional analysis is necessary to model the opposing direction.

[HCM Page 15-1]

Checklist and Index
Highway Capacity Manual Reference Guide

Signalized Intersections	✓	Page
Base Saturation Flow Rate Calibration		
Signal Operations		
Phase Duration		
Queue Storage Ratio		
Arrival Demand		
Multiple-Period Analysis		
Unsignalized Movements		
Planning Method		
Intersection Delay		
Saturation Flow Rate Adjustments		
Lane Groups		
Auxiliary Thru Lanes		
Signal Phasing		
Effective Green Time		
Critical v/c Ratio		
Back of Queue		
Average Delay		
Urban Streets	✓	Page
Level of Service		
Flow Profile		
Lane Blockage		
Sustained Spillback		
Access Points		
Upstream Filtering		
Flow Balancing		
RTOR Balancing		
Base Free-Flow Speed		
Arrival Type		
Optimized Timing		
Roundabout Corridors		
Travel Time Reliability		
Active Transportation Demand Modeling		
Interchange Ramp Terminals	✓	Page
Level of Service		
Experienced Travel Time (ETT)		

Signalized Intersections		
Lane Utilization		
Saturation Flow		
Downstream Queue		
Demand Starvation		
Effective Green Time		
Alternative Intersections/Interchanges		
Two-Way Stop Control	✓	Page
Calibration		
Two-Stage Gap Acceptance		
Upstream Signals		
Peak Hour Factor		
Rank 1 Delay		
Queuing		
Level of Service		
Average Delay		
All-Way Stop Control	✓	Page
Lane Utilization		
Lane Configuration		
Queuing		
Level of Service		
Average Delay		
Roundabouts	✓	Page
Calibration		
Capacity Models		
Lane Utilization		
Bypass Lane Definitions		
Yielding Bypass Lanes		
Non-Yielding Bypass Lanes		
Pedestrians		
Queuing		
Level of Service		
Average Delay		
Data		
Roundabout Corridors		
Basic Freeway Segments	✓	Page
Speed-Flow Curves		
Free-Flow Speed		
Lateral Clearance		

Truck Procedure		
Managed Lanes		
Adjustments		
Segments		
Travel Speed		
Break Points		
Ramp Density		
Driver Population		
One Direction		
Freeway Weaving Segments	✓	Page
Maximum Weaving Length		
Managed Lanes		
Adjustments		
Lane Changes		
Weaving Lanes		
Driver Population		
One Direction		
Freeway Merge & Diverge Segments	✓	Page
Acceleration/Deceleration Lanes		
Managed Lanes		
Adjustments		
Aggregated Density		
Adjacent Ramps		
Lane Additions/Drop		
Left-Hand Ramps		
Five-Lane Freeways		
Major Merge/Diverge		
Ramp Meters		
Driver Population		
One Direction		
Freeway Facilities	✓	Page
Calibration		
Oversaturated Conditions		
Travel Time Reliability		
Active Transportation Demand Management		
Managed Lanes		
Truck Procedure		
Work Zones		
Analysis Length		

Segments		
Weaving Segments		
Overlapping Segments		
Time-Space Domain		
Scale Factor		
Facility Capacity		
Weaving Capacity		
Capacity Adjustment		
Driver Population		
Bottlenecks		
Limitations		
Level of Service		
Multilane Highways	✓	Page
Speed-Flow Relationship		
Travel Speed		
Free-Flow Speed		
Access Points		
Lateral Clearance		
Break Points		
Truck Procedure		
Segments		
Level of Service		
Driver Population		
One Direction		
Two-Lane Highways	✓	Page
Highway Class		
Access Points		
Free-Flow Speed		
Passing Lanes		
Facilities		
One Direction		