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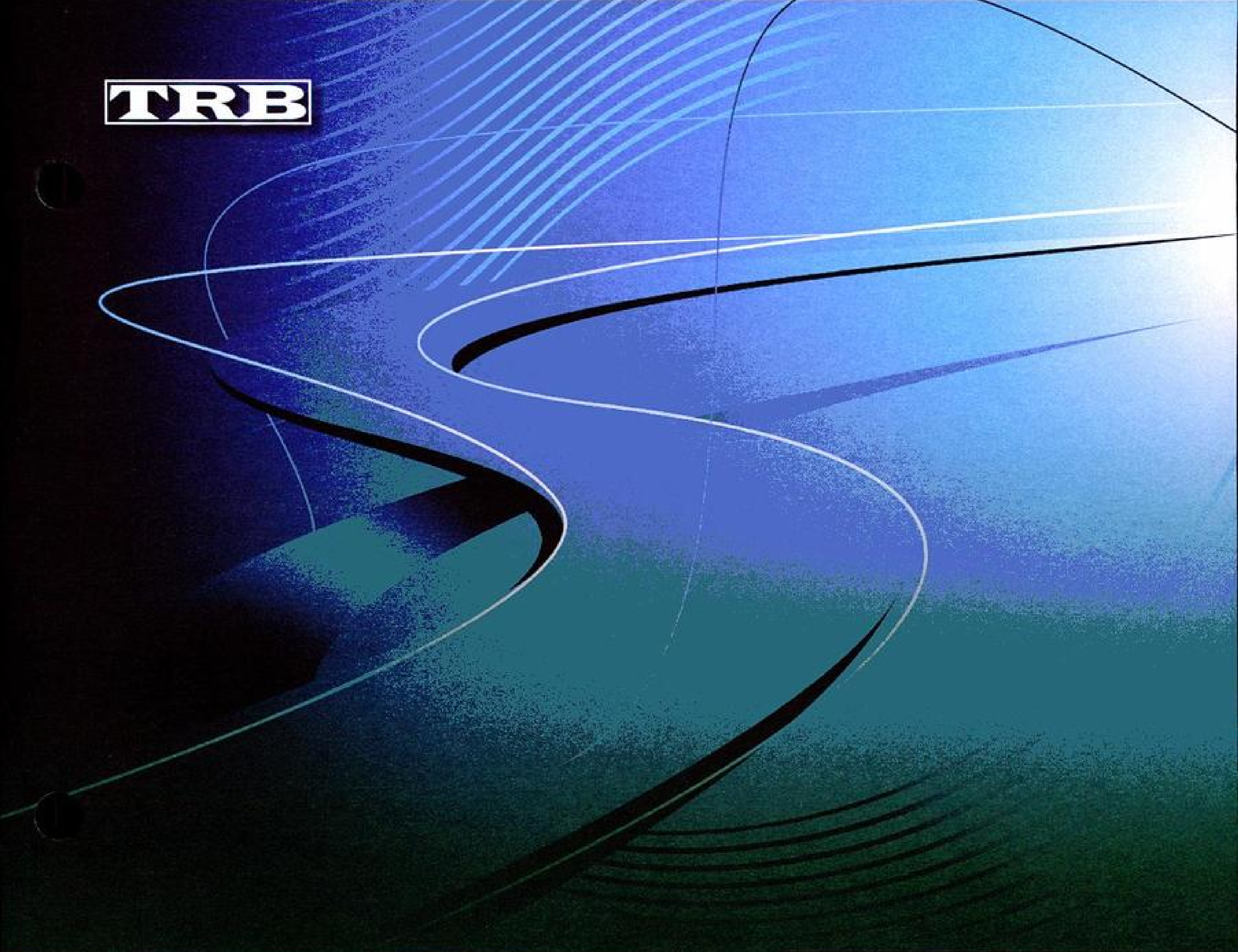

HIGHWAY CAPACITY MANUAL

6TH EDITION | A GUIDE FOR MULTIMODAL MOBILITY ANALYSIS

VOLUME 2: UNINTERRUPTED FLOW

The National Academies of
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CHAPTER 10
FREEWAY FACILITIES CORE METHODOLOGY

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

This chapter provides the core methodology for analyzing extended lengths of freeway composed of continuously connected basic freeway, weaving, merge, and diverge segments. Such extended lengths are referred to as a *freeway facility*. In this terminology, *facility* does not refer to an entire freeway from beginning to end; instead, it refers to a specific set of connected segments that have been identified for analysis. In addition, the term does not refer to a freeway system consisting of several interconnected freeways.

This chapter's methodology relies on the freeway segment methodologies in Chapters 12, 13, and 14. These methods focus on a single time period of interest, generally the peak 15 min within a peak hour. The methodology allows for the analysis of multiple and contiguous 15-min time periods and is capable of identifying breakdowns and the impact of such breakdowns over space and time. In essence, the methodology amalgamates hundreds or thousands of individual segment–time period analyses into a single facility analysis. It also allows for managed lanes and work zone analysis.

The methodology also is the basis of both freeway reliability analysis and the assessment of active traffic and demand management (ATDM) strategies. Both of these applications are described in Chapter 11, Freeway Reliability Analysis. Conceptually, Chapter 10 is a prerequisite for any reliability or ATDM analysis.

This chapter discusses the basic principles of the methodology and their application. Chapter 25, Freeway Facilities: Supplemental, provides a detailed description of all the algorithms that define the methodology. The methodology is integrated with the FREEVAL computational engine, which implements the complex computations involved. Volume 4 contains a user's guide to FREEVAL and an executable, research-grade software engine that implements the methodology.

CHAPTER ORGANIZATION

Section 2 presents the basic concepts of freeway facility operations, including definitions of analysis segments, capacity and free-flow speed concepts, and the level of service (LOS) framework for freeway facilities.

Section 3 presents the base methodology for evaluating freeway facilities, including details on all computational steps in the evaluation of a freeway facility.

Section 4 extends the core method presented in Section 3 to applications for managed lanes, including high-occupancy vehicle (HOV) and high-occupancy toll (HOT) lanes under various types of separation from the general purpose lanes. This method is based on the findings from National Cooperative Highway Research Program (NCHRP) Project 03-96 (*1–3*). Additional extensions include adaptations of the method for the evaluation of short-term and long-term work zones based on the findings from NCHRP Project 03-107 (*4, 5*).

Section 5 presents application guidance on using the results of a freeway facility analysis, including example results from the methods, information on the sensitivity of results to various inputs, and service volume tables.

RELATED HCM CONTENT

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

- Chapter 3, Modal Characteristics, where the Variations in Demand subsection for the automobile mode describes typical travel demand patterns for freeway and multilane highway segments;
- Chapter 4, Traffic Operations and Capacity Concepts, which provides background for the capacity and breakdown definitions specific to freeway and multilane highway segments that are presented in this chapter's Section 2;
- Chapter 11, Freeway Reliability Analysis, which provides extensions of the core freeway facility methodology for performing a whole-year reliability analysis and for assessing the whole-year impacts of ATDM strategies;
- Chapters 12, 13, and 14, which present the segment methodologies for basic freeway and multilane highway segments, freeway weaving segments, and freeway merge and diverge segments, respectively;
- Chapter 25, Freeway Facilities: Supplemental, which provides additional details for this methodology, including a detailed description of the oversaturated procedure, and a summary of the computational engine for freeway facility analysis;
- Chapter 26, Freeway and Highway Segments: Supplemental, which provides additional details for basic freeway segment capacity measurement and driver population factors;
- Case Study 4, New York State Route 7, in the HCM Applications Guide in Volume 4, which demonstrates how this chapter's methods can be applied to the evaluation of an actual freeway facility; and
- Section H, Freeway Analyses, in the *Planning and Preliminary Engineering Applications Guide to the HCM,* found in Volume 4, which describes how to incorporate this chapter's methods and performance measures into a planning effort.

2. CONCEPTS

OVERVIEW

A freeway is a separated highway with full control of access having two or more lanes in each direction dedicated to the exclusive use of motorized traffic. Freeway facilities are composed of various uniform segments that may be analyzed to determine capacity and LOS. Three types of segments are found on freeways:

- *Basic freeway segments* are all segments that are not merge, diverge, or weaving segments—whether general purpose or managed lanes. They are described in more detail in Chapter 12.
- *Freeway weaving segments* are segments in which two or more traffic streams traveling in the same general direction cross paths along a significant length of freeway without the aid of traffic control devices (except for guide signs). Weaving segments are formed when a diverge segment closely follows a merge segment or when a one-lane off-ramp closely follows a one-lane on-ramp and the two are connected by a continuous auxiliary lane. These segment types occur on both general purpose and managed lane facilities. In the latter case, and depending on the geometry, that segment could be labeled as a managed lane (ML) weaving or an ML access segment. Details for those designations are provided in Chapter 13.
- *Freeway merge and diverge segments* are segments in which two or more traffic streams combine to form a single traffic stream (merge) or a single traffic stream divides to form two or more separate traffic streams (diverge). These segment types occur on both general purpose and managed lane facilities. Details for those segments are provided in Chapter 14.

This chapter covers the core freeway facilities methodology, which may include managed lanes as part of the facility. The analysis is limited to a single study period not to exceed 24 h. Extensions of the core method to longer study periods are covered in Chapter 11, Freeway Reliability Analysis. Those extensions are intended to account for (*a*) the longer-term effects of both recurring (i.e., bottlenecks) and nonrecurring (e.g., due to weather, incidents, or work zones) congestion on freeway facility operations and (*b*) the effects of ATDM strategies in mitigating some of those negative impacts.

SECTIONS, SEGMENTS, AND INFLUENCE AREAS

Facilities Without Managed Lanes

The definitions of freeway sections and freeway segments and their respective influence areas should be clearly understood.

Sections are defined as extending from ramp gore point to gore point and are most directly compatible with the freeway performance databases used by many agencies. Some of these databases further distinguish between *internal sections* (e.g., between an off-ramp and an on-ramp at a diamond interchange) and *external sections* (between the final on-ramp at one interchange and the first off-ramp at the next downstream interchange). For the purpose of the HCM methodology, the distinction between internal and external sections is of no consequence. Sections are used in the planning-level application of this method detailed in Section 4 as well as for calibrating and validating freeway facilities, since sections are more directly compatible with field data than are HCM segments.

Segments are the portions of freeway sections corresponding to the definitions in the analysis methodologies presented in Chapters 12, 13, and 14. Segments can be identified by considering the area where a merge, diverge, or weave influences facility operations.

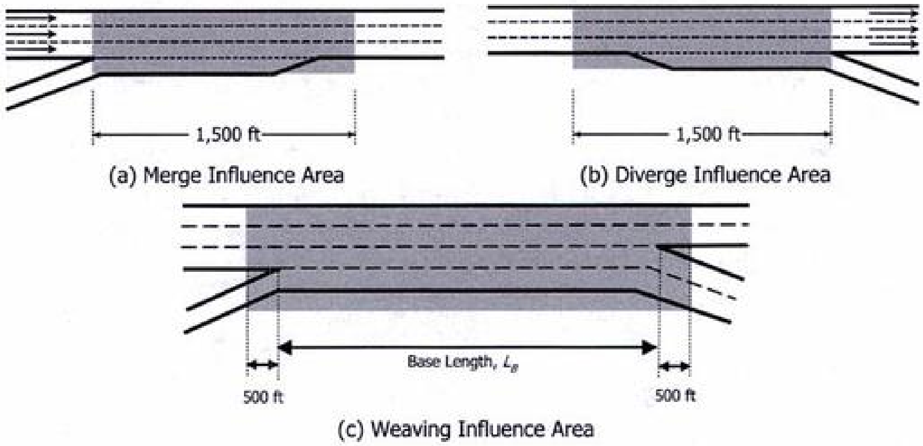
The *influence areas* of merge, diverge, and weaving segments are as follows:

- For *weaving segments,* the base length of the weaving segment plus 500 ft upstream of the entry point to the weaving segment and 500 ft downstream of the exit point from the weaving segment; entry and exit points are defined as the points where the appropriate edges of the merging and diverging lanes meet;
- For *merge segments,* from the point where the edges of the travel lanes of the merging roadways meet to a point 1,500 ft downstream of that point; and
- For *diverge segments,* from the point where the edges of the travel lanes of the diverging roadways meet to a point 1,500 ft upstream of that point.

Points where the "edges of travel lanes" meet are most often defined by pavement markings.

The influence areas of merge, diverge, and weaving segments are illustrated in Exhibit 10-1. A weaving segment is usually defined as the distance between the on-ramp and off-ramp gore points. However, its influence area extends 500 ft upstream and downstream of the gore-to-gore length as defined above.

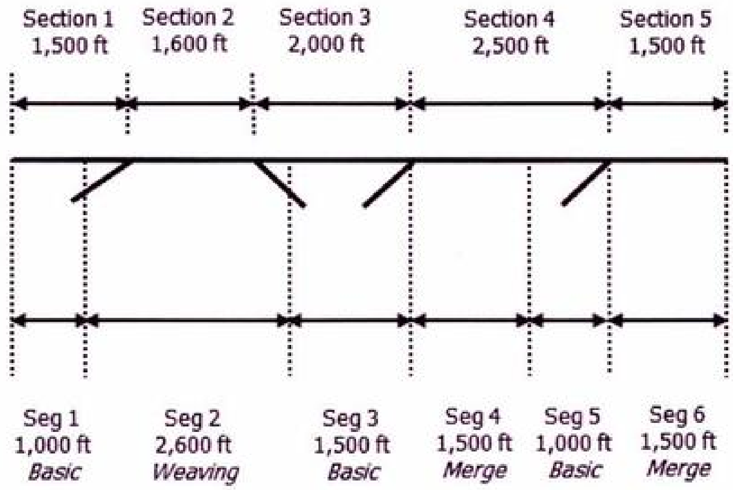
Exhibit 10-1
Influence Areas of Merge, Diverge, and Weaving Segments Without Managed Lanes



Basic freeway segments are any other segments along the freeway that are not within these defined influence areas. This does not imply that basic freeway segments are unaffected by the presence of nearby merge, diverge, and weaving segments. For example, the effects of a breakdown in a merge segment will propagate to both upstream and downstream segments, regardless of type. The impact of the frequency of merge, diverge, and weaving segments on the general operation of all segments is taken into account by the free-flow speed of the facility.

Basic freeway segments, therefore, exist even on urban freeways where merge and diverge points (most often ramps) are closely spaced. Exhibit 10-2 demonstrates this point by illustrating the definition of sections and segments. It shows a 9,100-ft (1.7-mi) length of freeway with four ramp terminals, two of which form a weaving segment. Overall, five sections are divided into six segments for consistency with the definition of HCM segments and their influence areas above.

Exhibit 10-2
Sections and Segments on an Urban Freeway



Even with an average ramp spacing of less than 0.5 mi, this length of freeway contains three basic freeway segments. The lengths of these segments are relatively short, but in terms of analysis methodologies they must be treated as basic freeway segments. Thus, while many urban freeways will be dominated by frequent merge, diverge, and weaving segments, there will still be segments classified and analyzed as basic freeway segments.

In applying the freeway facility methodology, the practice of beginning and ending the facility with a basic freeway segment is highly recommended. This segment may contain a *partial section*, since it does not both begin and end at a gore point. Sections 1 and 5 in Exhibit 10-2 are examples of partial sections. In comparing HCM results with field data, the analyst should consider that the length of the partial section will likely be less than the length of the section used in the database the field data came from.

The core methodology requires that all queues be contained within the facility. Thus, the first (basic) segment on the facility should be an upstream location that queues do not reach. Similarly, the last downstream (basic) segment should not be affected by queues spilling back from locations downstream of the defined facility.

Second, segment boundaries do not necessarily coincide with section boundaries. For example, the first weaving segment (Segment 2) in Exhibit 10-2 extends upstream and downstream of Section 2. The segment extends beyond the gore points, consistent with the definition of the weaving influence area in

Chapter 13. Because field data are likely reported to cover the extent of Section 2 (but not beyond), this difference represents a potential source of error in the calibration and validation effort.

In addition, HCM sections are homogeneous, but actual freeway sections are not necessarily homogeneous. For example, detectors are not necessarily matched with section definitions, resulting in either missing data points or locations with multiple sensors in one section. Guidance for field measurement and detector location is given in Chapter 26.

Facility Segmentation Guidance

Facility segmentation is only a small part of the overall freeway facility methodology, but it is highly important in ensuring the proper application of the methodology. Segmentation usually requires significant analyst time to ensure that the segment types and the computational procedures for those segments have been correctly entered. The segmentation step must be carried out by the analyst before any computations are performed.

This section provides guidance on segmenting facilities. However, the wide variety of configurations and conditions found on freeway facilities means that engineering judgment, beyond the guidelines specified here, may need to be applied in certain cases.

There are two basic steps in defining a freeway facility:

- Deciding appropriate facility termini and overall length, and
- Dividing the facility into HCM segments for analysis purposes.

For the first step, facility termini should be based on the following locations, which are provided in rank order (*6*):

1. Major freeway-to-freeway system interchanges;
2. Nonadjacent urbanized area boundaries;
3. Major intersecting (nonfreeway) routes;
4. Other special considerations such as major traffic generators (e.g., central city downtowns, airports) or state boundaries; and
5. Length, with consideration given to the type of area where the freeway is located, as well as the maximum facility length discussed in Section 3.

The rules above represent general guidance, but facilities may need to be extended or shortened to serve the purpose of the analysis. For example, as noted above, it is recommended that queues be contained within the defined facility if at all possible. When multiple consecutive facilities are analyzed, they should not overlap. A total facility length that is less than or equal to the distance that can be traversed within a 15-min analysis period at the free-flow speed is also recommended. If necessary, a longer study section can be subdivided into multiple smaller facilities for analysis.

The following general segmentation rules apply for the second step, dividing a facility into HCM segments:

- The first and last segments of the defined facility are recommended to be basic freeway segments.
- A new segment should be started whenever demand volume changes (i.e., at on- and off-ramps and at at-grade access points to managed lanes).
- A new segment should be started whenever capacity changes (i.e., when a full or auxiliary lane is added, when one or more lanes are added or dropped, when the terrain changes significantly, or where lane widths or lateral clearances change in a way that affects capacity).
- The influence area of a ramp is considered to be 1,500 ft, measured downstream from the gore point for on-ramps and upstream of the gore point for off-ramps. The end of a merge segment's ramp influence area often represents a transition to a basic freeway segment. Similarly, a basic segment transitions to a diverge segment at the beginning of the ramp influence area.
- Ramp segments, including the ramp influence area, are classified either as merge (on-ramp) or as diverge (off-ramp) segments, unless two adjacent merge and diverge segments are connected by an auxiliary lane, in which case the entire segment is coded as a weaving segment. In the latter case, the weave influence area extends 500 ft upstream and 500 ft downstream of the two respective gore areas (see Exhibit 10-2).
- When the gore-to-gore length between two adjacent merge and diverge segments exceeds 3,000 ft and no auxiliary lane exists, the section should be coded as a series of three segments (merge, basic, diverge). The basic segment length is the difference between the gore-to-gore spacing and 3,000 ft.
- When the gore-to-gore length of two adjacent merge and diverge segments is less than 3,000 ft but longer than 1,500 ft and no auxiliary lane exists, the section should be coded as a series of three segments, with the middle segment being defined as an overlap segment (merge, overlap, diverge). In this case, the overlap segment length is the difference between 3,000 ft and the gore-to-gore spacing, and the merge and diverge segment lengths are equal to the gore-to-gore spacing minus 1,500 ft.
- It is highly unusual to have ramp spacing (combinations of merge and diverge) less than 1,500 ft without the addition of an auxiliary lane to connect the two gore areas. However, when this occurs, the 1,500-ft merge or diverge segment length is truncated at the adjacent ramp gore point.
- Any remaining unassigned segments after all merge, diverge, weave, and overlap segments have been defined are labeled as basic segments.

Facilities with Managed Lanes

When managed lanes are present, additional managed lane segment types are defined, as explained in Chapters 12–14. Extensions to the methodology to address managed lanes are presented in Section 4 of this chapter.

Two important concepts related to managed lane segmentation are of interest:

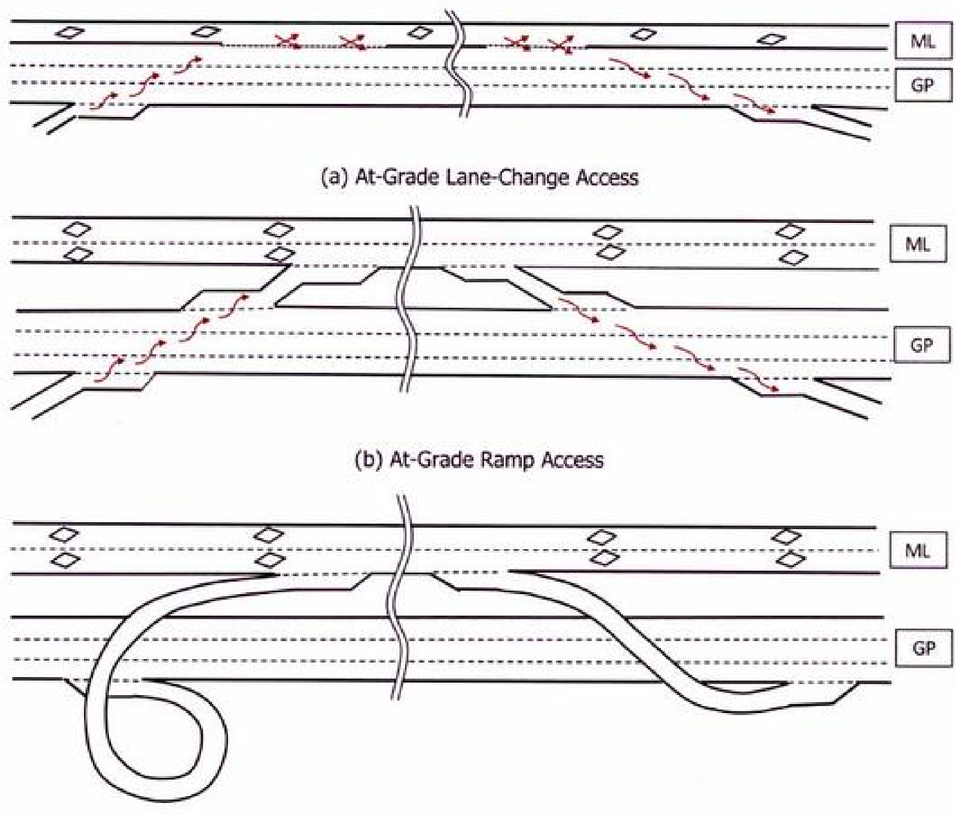
- The *lane group concept*, where each managed lane segment must be paired with an adjacent general purpose segment having the same length. This concept is explained in more detail in Section 4.
- The *friction* and *cross-weave concepts*, which describe how the general purpose and managed lane segments affect each other's performance. Friction in the managed lanes occurs at higher general purpose lane densities, when no physical separation is provided between the general purpose and managed lanes. Cross weave occurs when traffic from a general purpose on-ramp segment must cross multiple general purpose lanes to access the managed lane at a nearby ramp or access segment, thereby affecting general purpose segment capacity.

Access to and from a managed lane can occur in one of three ways, depicted in Exhibit 10-3 and described below:

- *At-grade lane-change access* occurs when managed lane traffic enters the general purpose lanes through a conventional on-ramp roadway (from the right), cross-weaves across multiple general purpose lanes, and enters the managed lane facility. Managed lane traffic exits in the same segment, so this configuration also results in a form of weaving movement. This access strategy is common for concurrent managed lane facilities. Access between managed and general purpose lanes is sometimes constrained to specific locations or openings, which affects the weaving intensity at these access points. This access configuration requires a *cross-weaving* movement across general purpose lanes for drivers to position themselves in advance of the access point and a *lane-change or weaving movement* to get from the general purpose lanes into the managed lanes.
- *At-grade ramp access* occurs where managed lane traffic enters the general purpose lanes through a conventional on-ramp roadway (from the right). Entering and exiting traffic may cross-weave across multiple general purpose lanes, similar to the first case, but the entrance to (or exit from) the managed lane facility is confined to an *at-grade* on-ramp or off-ramp. Operationally, the general purpose lanes may be affected by the cross-weaving flow. The managed lane operations, in turn, are only affected by merging and diverging maneuvers at the access points at the ramps. This access configuration requires a *cross-weaving* movement across the general purpose lanes for drivers to position themselves in advance of the access point and a *ramp merge movement* to get from the general purpose lanes into the managed lanes.
- *Grade-separated ramp access* occurs where the managed lanes are accessed on a grade-separated structure (i.e., bridge or underpass). The operational impact on the general purpose lanes is minimal in this case, because the cross-weaving movement is eliminated. The managed lanes are affected by friction from the entering or exiting ramp flows in the same fashion as the general purpose lanes. This access configuration does not require any cross-weaving across the general purpose lanes because of the grade-separated ramp, and the access to the managed lanes is handled by a

ramp merge movement. If all managed lane access is grade-separated, the result effectively is an entirely separate facility. However, a mix of grade-separated and at-grade access points is common.

Exhibit 10-3
Typology of Managed Lane Access Point Designs



(a) At-Grade Lane-Change Access

(b) At-Grade Ramp Access

(c) Grade-Separated Ramp Access

Notes: GP = general purpose, ML = managed lane.

The spatial extent of the *access point influence area* (APIA) for grade-separated ramp access is defined in the HCM's ramp merge and diverge methodology. The ramp influence area for general purpose facilities is defined to be 1,500 ft from the ramp gore for both on-ramps (measured downstream) and off-ramps (measured upstream). The APIA for grade-separated managed lane access points follows the same convention.

The intensity and impact of the *cross-weaving flows* between a general purpose ramp and the access region between the general purpose and managed lanes need to be analyzed for both at-grade access types. The minimum cross-weave length is defined as the distance between the closest upstream general purpose on-ramp gore and the start of the managed lane ramp or access opening [see for example, Exhibit 10-3(b)]. The maximum cross-weave length is defined as the distance from the ramp gore to the end of the access opening; this maximum does not apply to at-grade on-ramp access.

FREE-FLOW SPEED

Free-flow speed (FFS) is the average speed of vehicles on a given segment or facility, measured under low-volume conditions, when drivers are free to drive at their desired speed and are not constrained by the presence of other vehicles. FFS is considered the theoretical speed when both density and flow rate are zero. FFS is an important characteristic, since the capacity *c*, service flow rates *SF*, service volumes *SV*, and daily service volumes *DSV* depend on it.

Chapter 12, Basic Freeway Segments, presents speed–flow curves that indicate that, under base conditions, the FFS on freeways is expected to prevail at flow rates below 1,000 passenger cars per hour per lane (pc/h/ln). In this range, speed is insensitive to flow rate. This characteristic simplifies and allows measurement of free-flow speeds directly in the field and from sensor data.

However, there are exceptions where speeds are affected even at low flow rates. For example, speeds may be reduced if there is significant truck presence or if truck speeds are posted lower than passenger car speeds. Similarly, speeds on single-lane managed lane facilities have been shown to decline immediately even at low flow rates, rather than being stable until a breakpoint is reached. Under these conditions, the FFS becomes more of a theoretical concept that can be difficult to measure directly in the field.

Chapter 12 presents a methodology for estimating the FFS of a basic freeway segment if it cannot be measured directly. The FFS of a basic freeway segment is sensitive to three variables:

- Lane widths,
- Lateral clearances, and
- Total ramp density.

The most critical of these variables is total ramp density. *Total ramp density* is defined as the average number of on-ramp, off-ramp, major merge, and major diverge junctions per mile in the analysis direction (one side of the freeway only). Freeway interchanges can have two (standard diamond), three (partial cloverleaf), or four (full cloverleaf) ramps in the analysis direction. For segment analyses, ramp density is computed for a 6-mi section centered on the segment's midpoint; however, for facility analyses, ramp density is calculated across the entire facility (i.e., total number of ramps divided by total facility length).

While the methodology for determining FFS is provided in Chapter 12, Basic Freeway and Multilane Highway Segments, it is also applied in Chapter 13, Freeway Weaving Segments, and Chapter 14, Freeway Merge and Diverge Segments. Thus, FFS affects the operation of all basic, weaving, merge, and diverge segments on a freeway facility.

Exhibit 10-4 illustrates the determination of total ramp density on a 9-mi length of a directional freeway facility. As shown, four ramp terminals are located along the 9-mi facility; therefore, the total ramp density is 4/9 = 0.44 ramps/mi.

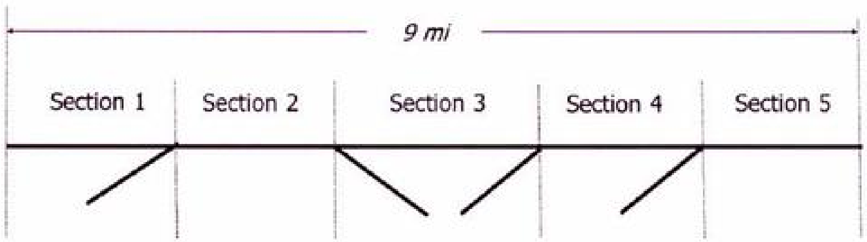


Exhibit 10-4
Sample Facility with Five Sections

FREEWAY FACILITY CAPACITY

Capacity traditionally has been defined for uniform segments of roadway, traffic, and control conditions. When facilities consisting of a series of connected segments are considered, the concept of capacity is more nuanced.

The methodologies of Chapters 12, 13, and 14 allow the capacity of each basic, weaving, merge, and diverge segment to be estimated. Since all segments of a facility are highly unlikely to have the same roadway, traffic, and control conditions and even less likely to have the same capacity, the freeway facility capacity hinges on the identification of the critical segment(s) where the breakdown starts. A definition based on this concept is provided below.

Conceptual Approach to Understanding Capacity and Flow Regimes on a Freeway Facility

Consider the sample facility shown in Exhibit 10-4 and the associated data in Exhibit 10-5 below. This example illustrates five consecutive sections that are to be analyzed as one "freeway facility." Note that while this conceptual example is illustrated by using sections, the methodology in fact performs computations at the segment level.

Exhibit 10-5
Example of the Effect of Capacity on Demand and Actual Flow Rates on a Freeway Facility

Scenario	Performance Measures	Freeway Section 1	2	3	4	5
Scenario 1 (stable flow)	Demand v_d (veh/h)	3,400	3,500	3,400	4,200	4,400
	Capacity c (veh/h)	4,000	4,000	4,500	4,500	4,500
	Volume v_a (veh/h)	3,400	3,500	3,400	4,200	4,400
	v_d/c ratio	0.850	0.875	0.756	0.933	0.978
	v_a/c ratio	0.850	0.875	0.756	0.933	0.978
Scenario 2 (add 200 veh/h to each section)	Demand v_d (veh/h)	3,600	3,700	3,600	4,400	4,600
	Capacity c (veh/h)	4,000	4,000	4,500	4,500	4,500
	Volume v_a (veh/h)	3,600	3,700	3,600	4,400	4,500
	v_d/c ratio	0.900	0.925	0.800	0.978	1.022
	v_a/c ratio	0.900	0.925	0.800	0.978	1.000
Scenario 3 (increase Scenario 1 demand by 10% in all sections)	Demand v_d (veh/h)	3,740	3,850	3,740	4,620	4,840
	Capacity c (veh/h)	4,000	4,000	4,500	4,500	4,500
	Volume v_a (veh/h)	3,740	3,850	3,740	4,500	4,500
	v_d/c ratio	0.935	0.963	0.831	1.027	1.076
	v_a/c ratio	0.935	0.963	0.831	1.000	1.000

Note: Shaded cells indicate segments where demand exceeds capacity.

Demand flow rates v_d, capacities c, and actual served (or observed) flow rates v_a are given, as are the resulting v_d/c and v_a/c ratios. An increase in capacity is observed in Sections 3 to 5. The example covers three conceptual scenarios that illustrate the difference between *demand flow rate* and *actual served flow rate*, as a result of section capacity constraints that meter the full demand.

In Scenario 1, none of the demand flow rates exceeds the section capacities on the facility. Thus, no breakdowns occur, and the actual volume served is the same as the demand flow rates (i.e., $v_d = v_a$ for this scenario). None of the v_d/c ratios exceeds 1.00, although relatively high ratios (0.978) occur in Section 5. Operating conditions in scenarios such as Scenario 1 imply a stable flow regime.

Scenario 2 adds 200 veh/h of through demand to each section. In this case, Section 5 experiences a breakdown since its demand flow rate exceeds its capacity. In this section, the actual flow rate v_a differs from the demand flow rate v_d, since the actual flow rate can never exceed the section capacity c.

In Scenario 3, all demand flow rates in Scenario 1 are increased by 10%. In this case, the demand flow rate exceeds capacity in both Sections 4 and 5. Again, the demand and actual flow rates will differ in these sections. Operating conditions under Scenario 2 or 3 are considered to be unstable, with breakdown and propagation of queuing conditions upstream likely to occur.

This example highlights a number of important points concerning the analysis of freeway facilities:

1. In applications of the methodology, it is critical that the difference between *demand* flow rate v_d and *actual* flow rate v_a be highlighted and that both values be clearly and appropriately labeled. The actual flow rate can never exceed the section or segment capacity.
2. It might be argued that the analysis of Scenario 1 above is sufficient to understand the facility's operation as long as all its segments are undersaturated (i.e., all sections' v_d/c ratios are less than or equal to 1.00). However, when *any section's* v_d/c ratio exceeds 1.00, such a simple analysis ignores the propagation of queues in space and time.
3. The analysis shown in Exhibit 10-5 for Scenarios 2 and 3 is incomplete. In Scenario 2, when Section 5 breaks down, queues will begin to form and propagate upstream. Thus, even though the demands in Sections 1 through 4 are less than those segments' capacities, the queues generated by Section 5 will propagate over time, possibly all the way to Segment 1, and thus could significantly affect upstream operations. In Scenario 3, Sections 4 and 5 fail. Their queues will also propagate upstream over time and alter the actual flow rates v_a on the affected segments.
4. In Scenarios 2 and 3, sections downstream of Section 5 are also affected, since the demand flow rate is prevented from reaching those sections by the capacity constraint on Section 5 (and Section 4 in Scenario 3).
5. In this example, the sections that break down first do not necessarily have the lowest capacities. Breakdown occurs first in Section 5, which has one of the higher capacities on the facility but also the highest flow rate and, therefore, the highest demand-to-capacity ratio.

Capacity in the Context of Freeway Facilities

In view of all these issues, the notion of capacity on a freeway facility can be described as follows:

> *Freeway facility capacity is governed by the position and severity of active bottlenecks (i.e., segments with $v_d/c > 1.0$) along its length. Both characteristics vary over time and space, depending on the time-varying demand flow rates on each facility segment. A bottleneck that is active at one time may hide another (less severe) bottleneck further downstream by suppressing demand flows to that downstream bottleneck. In short, there is no simple definition for freeway facility capacity, other than it is variable over time and influenced by the timing and location of active bottlenecks.*

The *critical segment* is generally defined as the bottleneck segment that will break down the earliest, given that all traffic, roadway, and control conditions do not change, including the spatial distribution of demands on each component segment. This definition is not a simple one. It depends on the relative demand characteristics and, as stated earlier, *can change over time* as the demand pattern changes. Facility capacity may be different from the capacity of the component segment with the lowest capacity. Therefore, the evaluation of individual segment demands and capacities is important. In fact, the methodology explained later in this chapter specifies that the facility be assigned LOS F in any time interval in which any segment demand-to-capacity ratio exceeds 1.00.

Active and Hidden Bottlenecks

The freeway facilities methodology is able to identify both *active* and *hidden* bottlenecks. An *active bottleneck* is defined as a segment with a demand-to-capacity ratio (v_d/c) greater than 1.0, an actual flow-to-capacity ratio (v_a/c) equal to 1.0, and queuing upstream of the bottleneck segment. Active bottlenecks are the congestion points best known to operating agencies and are of critical importance in validating the procedure to match field-observed conditions. The actual flow at an active bottleneck is metered by its capacity, resulting in downstream segments likely having v_a/c ratios that are less than their v_d/c ratios.

A *hidden bottleneck* is defined as a segment with a demand-to-capacity ratio (v_d/c) greater than 1.0 but an actual flow-to-capacity ratio (v_a/c) typically less than 1.0 (or equal to 1.0 in some cases), with no queues forming upstream of the segment. In other words, hidden bottlenecks are segments with v_d/c greater than 1.0 where the demand is metered by a more severe active bottleneck upstream. Since a portion of the true segment demand is stored in the upstream queue, the actual flow arrivals at the hidden bottleneck may be less than 1.0 and no queues are formed.

Knowledge of hidden bottlenecks is of primary importance when improvement strategies for a congested facility are evaluated. For example, if an analysis points to an active bottleneck, the operating agency may decide to improve operations by widening the bottleneck segment. However, if one or more hidden bottlenecks are located downstream of that segment, the improvements may simply result in congestion migrating downstream. For true removal of the congestion, an agency may need to improve all active and hidden bottlenecks.

Prebreakdown and Queue Discharge Capacity

The term *capacity*, as used thus far, refers to the critical segment capacity—the flow rate that immediately precedes the onset of the breakdown. Chapter 12 defines breakdown as a sudden drop in speed of at least 25% below the free-flow speed for a sustained period of at least 15 min that results in queuing upstream of the bottleneck. Thus, the segment capacities shown in Exhibit 10-5 are also called *prebreakdown* capacities or flow rates, defined as the 15-min average flow rate immediately preceding the breakdown event.

Once the breakdown takes place and queues begin to form, the flow rates discharging from the queue at the bottleneck are generally lower than the prebreakdown capacity. This postbreakdown flow rate or *queue discharge flow rate* is defined as the 15-min flow rate during oversaturated conditions (i.e., during the time interval after breakdown and before recovery). The difference in flow rate varies considerably in the research literature, from a value of zero (and sometimes negative values) up to 15% to 20% (*7*). The amount of the drop was also found to depend on the magnitude of the prebreakdown flow rate.

A synthesis of the literature indicates that an average value for the capacity drop is about 7% (*7*). This reduced capacity is used in the oversaturated analysis procedure to estimate the rate at which queues will form and dissipate once demand exceeds capacity. When the queue is cleared, the segment's original (prebreakdown) capacity is restored. Details of the two-capacity phenomenon, and its application in the core computational methodology, are explained in the next section. A formal definition of freeway segment capacity is provided in Chapters 12, and a measurement method is provided in Chapter 26.

LOS: COMPONENT SEGMENTS AND THE FREEWAY FACILITY

LOS of Component Segments

Chapters 12, 13, and 14 provide methodologies for determining the LOS in basic, weaving, merge, and diverge segments on the basis of the segment's average density. In all cases, LOS F is also defined when v_d/c is greater than 1.00.

This chapter's methodology provides an analysis of breakdown conditions, including the spatial and time impacts of a breakdown. Thus, in the performance of a facility-level analysis, LOS F in a component segment can be identified (*a*) when the segment v_d/c is greater than 1.00 and (*b*) when a queue resulting from a downstream breakdown extends into an upstream segment. The latter cannot be estimated by using the individual segment analysis procedures of Chapters 12–14.

Thus, when a facility-level analysis is performed, LOS F for a component segment will be identified in two complementary ways:

- When v_d/c is greater than 1.00 for one or more critical segments, or
- When the segment density is greater than 45 pc/mi/ln for basic freeway segments or 43 pc/mi/ln for weaving segments.

The latter condition identifies segments in which queues have formed as a result of downstream breakdowns.

LOS for a Freeway Facility

Because LOS for basic, weaving, merge, and diverge segments on a freeway is defined in terms of density, LOS for a freeway facility is also defined on the basis of density. The method distinguishes between density thresholds used to designate LOS on urban and rural freeway facilities on the basis of research (*8*). Such a distinction in LOS is not made at the segment level.

The classification of a facility as urban or rural is made on the basis of the Federal Highway Administration's smoothed or adjusted urbanized boundary definition (*9*), which in turn is derived from Census data. Facilities that fall fully within an urban area or fully outside of it are classified as urban or rural, respectively. If a freeway facility crosses an urbanized area boundary, analyst judgment is needed in classifying it as urban or rural. Generally, the entire length of the facility needs to be assigned the same area type.

A facility analysis will result in a density determination and LOS for each component segment. The facility LOS will be based on the weighted average density for all segments within the defined facility. Weighting is done on the basis of segment length and number of lanes in each segment, in accordance with Equation 10-1:

$$D_F = \frac{\sum_{i=1}^{n} D_i \times L_i \times N_i}{\sum_{i=1}^{n} L_i \times N_i}$$

Equation 10-1

where

D_F = average density for the facility in a given 15-min analysis period (pc/mi/ln),

D_i = density for segment i (pc/mi/ln),

L_i = length of segment i (mi),

N_i = number of lanes in segment i, and

n = number of segments in the defined facility.

LOS criteria for urban and rural freeway facilities are shown in Exhibit 10-6. Urban LOS thresholds are the same density-based criteria used for basic freeway segments. Studies on LOS perception by rural travelers indicate the presence of lower-density thresholds in comparison with their urban freeway counterparts. The average LOS applies to a specific time period, usually 15 min.

Exhibit 10-6
LOS Criteria for Urban and Rural Freeway Facilities

	Freeway Facility Density (pc/mi/ln)	
LOS	**Urban**	**Rural**
A	≤11	≤6
B	>11–18	>6–14
C	>18–26	>14–22
D	>26–35	>22–29
E	>35–45	>29–39
F	>45 or any component segment v_d/c ratio > 1.00	>39 or any component segment v_d/c ratio >1.00

A LOS descriptor for the overall freeway facility must be used with care. The LOS for individual segments composing the facility should also be reported. The overall LOS, being an average, may mask serious problems in individual segments of the facility.

This is particularly important if one or more of the component segments are operating at LOS F. As described in this chapter's methodology section, the freeway facility methodology applies models to estimate the propagation of the effects of a breakdown in time and space. Where breakdowns occur in one or more segments of a facility, the average LOS is of limited use.

For urban freeway facilities, LOS A through E are defined on the basis of the same densities that apply to basic freeway segments. Rural freeway facilities have lower density thresholds, as indicated in Exhibit 10-6. This difference is a result of rural motorists' higher LOS expectations. The analyst is cautioned that a rural facility analysis may produce LOS F without any of its segments experiencing breakdown ($v_d/c > 1$). As a result, LOS F for a facility represents a case in which any component segment of the freeway has a v_d/c ratio that exceeds 1.00 or in which the average density of the study facility exceeds 45 pc/mi/ln (for urban freeways) or 39 pc/mi/ln (for rural freeways). This chapter's methodology allows the analyst to map the impacts of breakdowns in time and space, and close attention to the LOS of component segments is necessary.

3. METHODOLOGY

This chapter's methodology provides for the integrated analysis of a freeway facility composed of connected segments. The methodology builds on the models and procedures for individual segments described in Chapter 12, Basic Freeway and Multilane Highway Segments; Chapter 13, Freeway Weaving Segments; and Chapter 14, Freeway Merge and Diverge Segments.

SCOPE OF THE METHODOLOGY

Because the freeway facility methodology builds on the segment methodologies of Chapters 12, 13, and 14, it incorporates all aspects of those chapters' methodologies. This chapter's method adds the ability to analyze operations when LOS F exists on one or more segments of the study facility. It draws from research sponsored by the Federal Highway Administration (*10*).

In Chapters 12–14, the existence of a breakdown (LOS F) is identified for a given segment, as appropriate. However, the segment methodologies do not provide tools for analyzing the impacts of such breakdowns over time and space.

The methodology analyzes a set of connected segments over a set of sequential 15-min periods. In deciding which segments and time periods to analyze, two principles should be observed:

1. The first and last segments of the defined facility should *not* operate at LOS F.
2. The first and last time periods of the analysis should *not* include any segments that operate at LOS F.

When the first segment operates at LOS F, a queue extends upstream that is not included in the facility definition and that therefore cannot be analyzed. The first segment should thus be long enough to contain the queue, although this may not always be practical or possible. When a queue does extend beyond the first segment, the methodology reports the number of unserved vehicles that should be considered by the analyst.

When the last segment operates at LOS F, there may be a downstream bottleneck outside the facility definition. Again, the impact of this congestion cannot be evaluated because it is not contained within the defined facility. LOS F during either the first or the last time period creates similar problems with regard to time. If the first time period operates at LOS F, LOS F may exist in previous time periods as well. If the last time period is at LOS F, subsequent periods may also operate at LOS F. The impact of a breakdown cannot be fully analyzed unless the queuing is contained within the defined facility and defined analysis period. The same problems would exist if the analysis were performed by using simulation.

Spatial and Temporal Limits

There is no limit to the number of time periods that can be analyzed. The temporal extent should be sufficiently long to contain the formation and dissipation of all queues as discussed above. Ideally, 30 min of analysis time should be added before and after the known peak period for a clear picture of the onset and dissipation of congestion.

The length of the freeway should be less than the distance a vehicle traveling at the average speed can achieve in 15 min. This specification generally results in a maximum facility length between 9 and 12 mi. Facilities longer than these limits should be divided into subfacilities at appropriate breakpoints. Each subfacility can then be analyzed separately with this chapter's procedure.

Performance Measures

The core freeway facility methodology generates the following performance measures for each segment and time period being evaluated:

- Capacity,
- FFS,
- Demand-to-capacity and volume-to-capacity ratios,
- Space mean speed,
- Average density,
- Travel time (min/veh),
- Vehicle miles traveled (VMT, demand and volume served),
- Vehicle hours of travel (VHT),
- Vehicle hours of delay (VHD), and
- Motorized vehicle LOS for each component segment and for the facility.

In addition, space mean speed, average density, travel time, VMT, VHT, VHD, and LOS are aggregated in each time interval across all segments in the facility. Performance measures are not aggregated across time periods. Details on how this aggregation is performed are given in Chapter 25.

Strengths of the Methodology

The following are strengths of the freeway facilities methodology:

1. The methodology captures oversaturated and undersaturated conditions in an extended time–space domain.
2. The methodology accounts for all active and hidden mainline bottlenecks and can be used to evaluate the operational effects of control strategies and capacity improvements along the facility.
3. The methodology explicitly tracks queues as they form and dissipate across segments and time intervals.
4. The methodology allows for time-varying demands and capacities, thereby permitting the evaluation of control strategies that affect demand (e.g., traffic diversion or peak spreading) or capacity (e.g., hard running shoulders, lane additions, ramp metering).
5. The methodology can account for the effects of short-term incidents, weather events, and work zones.

6. The methodology is consistent with the segment methodologies in Chapters 12, 13, and 14 if all v_d/c ratios are less than or equal to 1.0, and it properly accounts for the interaction of segments when any v_d/c ratio is greater than 1.0.

Given enough time, a completely undersaturated time–space domain can be analyzed manually, although the process can be difficult and time-consuming. It is not expected that manual analysis of a time–space domain that includes oversaturation will ever be carried out. A computational engine is needed to implement the methodology, regardless of whether the time–space domain contains oversaturated segments and time periods. The engine is available in the online HCM Volume 4 for research purposes but should not be used for commercial applications.

Limitations of the Methodology

The methodology has the following limitations:

1. The methodology does not account for delays caused by vehicles using alternative routes or vehicles leaving before or after the analysis period.
2. Multiple overlapping breakdowns or bottlenecks are difficult to analyze and cannot be fully evaluated by this methodology. Other tools may be more appropriate for specific applications beyond the capabilities of the methodology. Consult Chapter 6, HCM and Alternative Analysis Tools, for a discussion of simulation and other models.
3. Spatial, temporal, modal, and total demand responses to traffic management strategies are not automatically incorporated into the methodology. On viewing the facility traffic performance results, the analyst can modify the demand input manually to analyze the effect of user-demand responses and traffic growth. The accuracy of the results depends on the accuracy of the estimation of user-demand responses.
4. The completeness of the analysis will be limited if any freeway segment in the first or last time interval, or the first or last freeway segment in any time period, has a demand-to-capacity ratio greater than 1.00.
5. The method does not address conditions in which off-ramp capacity limitations result in queues that extend onto the freeway or affect the behavior of off-ramp vehicles.
6. Because this chapter's methodology incorporates the methodologies for basic, weaving, merging, and diverging freeway segments, the limitations of those procedures also apply here.
7. The method does not include analysis of the streetside terminals of freeway on- and off-ramps. The methodologies of Chapters 19, 20, 21, and 22 should be used for intersections that are signalized, two-way STOP-controlled, all-way STOP-controlled, and roundabouts, respectively. Chapter 23, Ramp Terminals and Alternative Intersections, provides a more comprehensive analysis of freeway interchanges where the streetside ramp terminals are signalized intersections or roundabouts.

REQUIRED DATA AND SOURCES

The analysis of a freeway facility requires details concerning each segment's geometric characteristics, as well as each segment's demand characteristics during each analysis time period. Exhibit 10-7 shows the data inputs that are required for an operational analysis of a freeway facility, potential sources of these data, and suggested default values.

Exhibit 10-7
Required Input Data, Potential Data Sources, and Default Values for Freeway Facility Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Free-flow speed (mi/h)	Direct speed measurements, estimate from FFS prediction algorithm	Base free-flow speed: speed limit + 5 mi/h (range 55–75 mi/h)
Segment and section length (ft)	Road inventory, aerial photo	Must be provided
Number of mainline freeway lanes (one direction)	Road inventory, aerial photo	At least 2
Lane width (ft)	Road inventory, aerial photo	12 ft (range 10–12 ft)
Right-side lateral clearance (ft)	Road inventory, aerial photo	6 ft (range 0–6 ft)
Total ramp density in analysis direction	Road inventory, aerial photo	Must be provided (range 0–6 ramps/mi)
Area type (urban, rural)	Road inventory, aerial photo	Must be provided
Terrain type (level, rolling, specific grade)	Design plans, analyst judgment	Must be provided
Ramp number of lanes	Road inventory, aerial photo	1 lane
Ramp acceleration or deceleration lane length (ft)	Road inventory, aerial photo	500 ft
Ramp free-flow speed (mi/h)	Road inventory, aerial photo	35–45 mi/h
Geometry of managed lanes	Road inventory, aerial photo	Must be provided
	Demand Data	
Mainline entry demand volume by time interval (veh/h)	Field data, modeling	Must be provided
On-ramp and off-ramp demands by time interval (veh/h)	Field data, modeling	Must be provided
Weaving demands on weaving segments by time interval (veh/h)	Field data, modeling	Must be provided
Heavy vehicle percentage (%)	Field data	5% (urban), 12% (rural)[a]
Driver population speed and capacity adjustment factors (decimal)	Field data	1.00 (see Chapter 26 for details)
Jam density (pc/mi/ln)	Field data	190 (range 150–270)
Queue discharge capacity drop (%)	Field data	7% (range 0%–20%)
Managed lane demand volume (veh/h)	Field data, modeling	Must be provided

Notes: ***Bold italic*** indicates high sensitivity (>20% change) of service measure to the choice of default value.
Bold indicates moderate sensitivity (10%–20% change) of service measure to the choice of default value.
[a] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.

Where any data item is not readily available or collectible, the analysis may be supplemented by using consistent default values for each segment. Lists and discussions of default values are found in Chapters 12, 13, and 14 for basic freeway, weaving, and merge and diverge segments, respectively.

OVERVIEW

The freeway facility methodology represents one of three parts in an evaluation sequence that can also include a freeway reliability analysis and an evaluation of ATDM strategies. Part A: Core Freeway Facility Analysis (single study period) is presented in this chapter, while Parts B and C are presented in Chapter 11, Freeway Reliability Analysis. Part A constitutes the core methodology; Parts B and C represent adaptations and extensions of the methodology for reliability and ATDM assessment, respectively. On completion of the 17 computational steps in the core methodology, the analyst may decide to continue to perform reliability and ATDM analyses by using the procedures described in Chapter 11.

Exhibit 10-8 summarizes the process of implementing the core methodology for analyzing freeway facilities. The methodology adjusts vehicle speeds appropriately to account for the impacts of adjacent upstream or downstream segments. The methodology can analyze freeway traffic management strategies only in cases where 15-min intervals (or their multiples) are appropriate and when reliable data for estimated capacity and demand exist.

COMPUTATIONAL STEPS

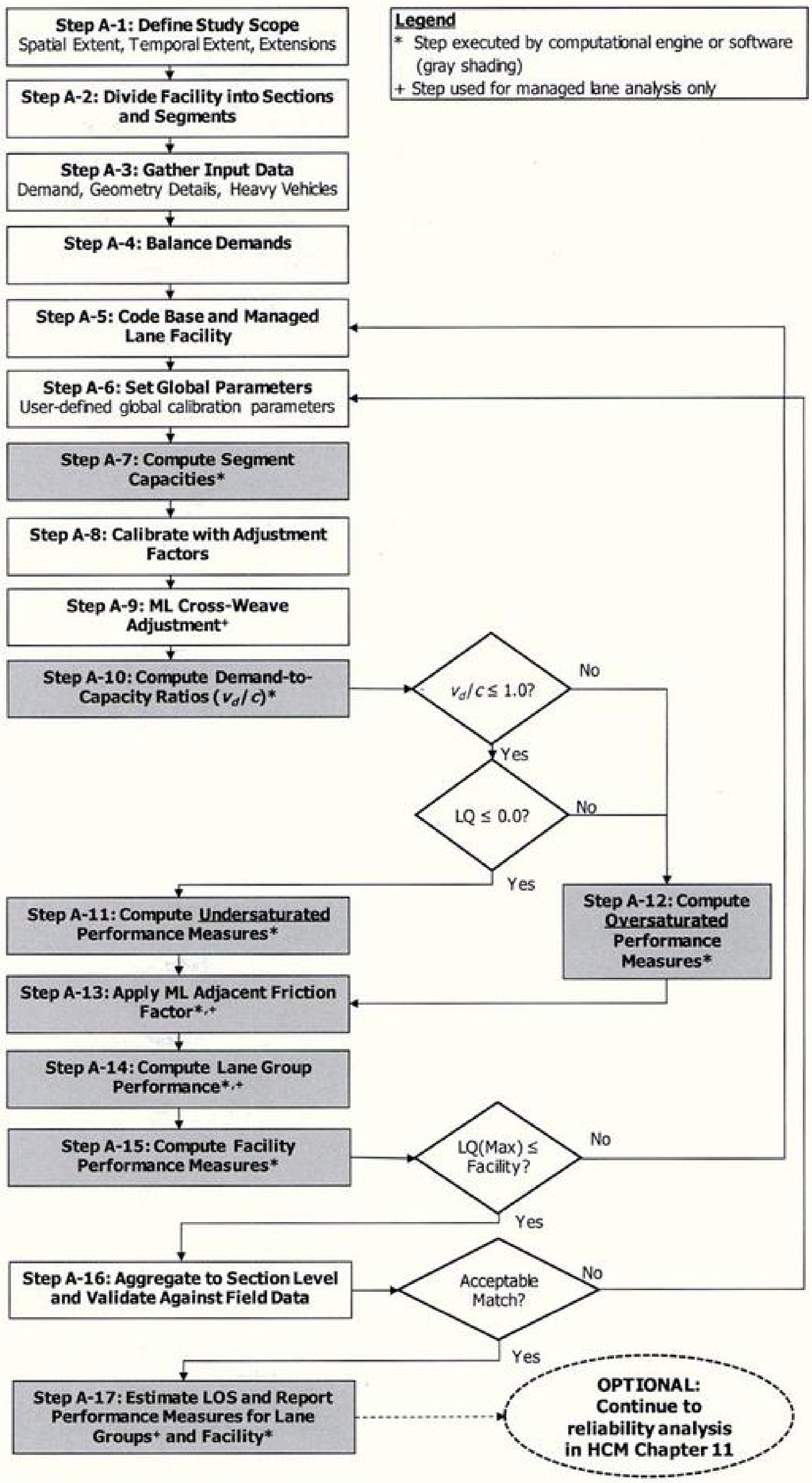
This section describes the methodology's computational modules. To simplify the presentation, the focus is on the function of and rationale for each module. Chapter 25 presents an expanded version of this section, including all the supporting analytical models and equations.

Step A-1: Define Study Scope

In this initial step, the analyst defines the spatial extent of the facility (start and end points, total length) and the temporal extent of the analysis (number of 15-min analysis periods). The analyst should further decide which study extensions (if any) apply to the analysis (i.e., managed lanes, reliability, ATDM).

A time–space domain for the analysis must be established. The domain consists of a specification of the freeway *sections* and *segments* included in the defined facility and an identification of the time intervals for which the analysis is to be conducted. For the freeway facility shown in Exhibit 10-9, a typical time–space domain is shown in Exhibit 10-10.

Exhibit 10-8
Freeway Facility Methodology



Note: ML = managed lane, LQ = length of queue.

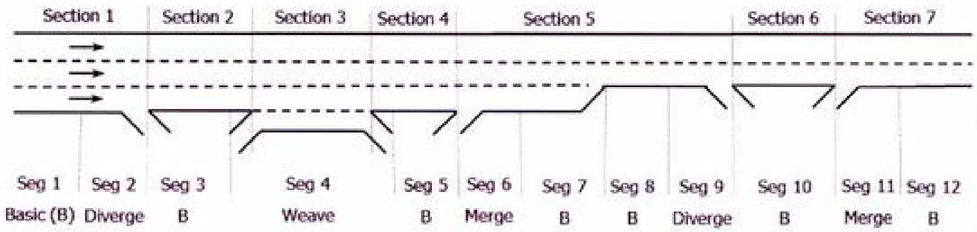


Exhibit 10-9
Example Freeway Facility for Time–Space Domain Illustration

Exhibit 10-10
Example Time–Space Domain for Freeway Facility Analysis

Analysis Period	Seg 1	Seg 2	Seg 3	Seg 4	Seg 5	Seg 6	Seg 7	Seg 8	Seg 9	Seg 10	Seg 11	Seg 12
1												
2												
3												
4												
5												
6												
7												
8												

Note: Seg = segment.

The horizontal scale indicates the distance along the freeway facility. A freeway *section* boundary occurs where there is a change in demand—that is, at each on-ramp or off-ramp. These areas are referred to as *sections* because adjustments will be made within the procedure to determine where *segment* boundaries should be for analysis. This process relies on the influence areas of merge, diverge, and weaving segments, discussed earlier in this chapter, and on variable length limitations specified in Chapter 13 for weaving segments and in Chapter 14 for merge and diverge segments. The facility in Exhibit 10-9 corresponds to seven sections that are then divided into 12 segments. The time–space domain is illustrated at the segment level, which is the basis of the HCM methodology. However, aggregation to the section level is possible and may be needed to compare the results with field data available only at the section level.

The longer the facility length without congestion on the horizontal scale, the more the congested travel times are diluted (see Equation 10-1). The analyst should avoid overly long segments at the edges of the space domain when possible, to avoid diluting the overall results. However, the first segment should be long enough to contain all queues, if practical.

The vertical scale indicates the study duration. Time extends down the time–space domain, and the scale is divided into 15-min intervals. In the example shown, there are 12 segments and 8 analysis periods, yielding 12 × 8 = 96 time–space *cells*, each of which will be analyzed within the methodology. The analysis could be extended to up to a 24-h analysis, corresponding to ninety-six 15-min analysis periods.

The boundary conditions of the time–space domain are extremely important. The time–space domain will be analyzed as an independent freeway facility having no interactions with upstream or downstream portions of the freeway or with any connecting facilities, including other freeways and surface facilities. Therefore, no congestion should occur along the four boundaries of the time–space domain. The cells located along the four boundaries should all have demands less than capacity and should contain undersaturated flow conditions.

A proper analysis of congestion within the time–space domain can occur only if the congestion is limited to internal cells not along the time–space boundaries. If necessary, the analysis domain should be extended in time, space, or both to contain all congestion effects.

Step A-2: Divide Facility into Sections and Segments

In this step, the analyst first defines the number of sections from gore point to gore point along the selected facility. These gore-to-gore sections are more consistent with modern freeway performance databases than are HCM segments, and this consistency is critical for calibrating and validating the freeway facility.

The analyst later divides sections into HCM segments (basic, merge, diverge, weave, overlapping ramp, or managed lane segment) as described below. Judgment may be needed for segments that do not cleanly fit the HCM definitions. The first and last segment must always be a basic segment, and these may be considered as "half sections," since only one gore point is included in each. This point was illustrated previously in Exhibit 10-2 and Exhibit 10-9.

When a facility includes managed lanes, this step also includes defining parallel lane groups for managed lanes and general purpose lanes, as will be described in Section 4.

The sections of the defined freeway facility are bounded by points where demand changes. However, this approach does *not* fully describe individual *segments* for analysis within the methodology. The conversion from sections to analysis segments can be performed manually by applying the principles discussed here, along with those given previously in the Facility Segmentation Guidance subsection of Section 2.

Chapter 14, Freeway Merge and Diverge Segments, indicates that each merge segment extends from the merge point to a point 1,500 ft downstream of it. Each diverge segment extends from the diverge point to a point 1,500 ft upstream of it. This allows for a number of scenarios affecting the definition of analysis segments within the defined freeway.

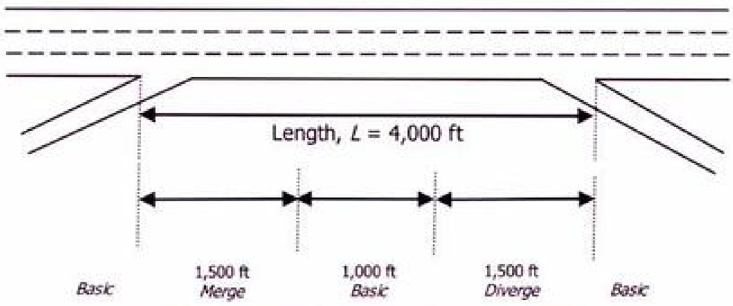
Consider the illustration of Exhibit 10-11. It shows a one-lane on-ramp followed by a one-lane off-ramp with no auxiliary lane between them. The illustration assumes that there are no upstream or downstream ramps or weaving segments that impinge on this section.

In Exhibit 10-11(a), the two ramps are 4,000 ft apart. The merge segment extends 1,500 ft downstream from the on-ramp while the diverge segment extends 1,500 ft upstream from the off-ramp, which leaves a 1,000-ft basic freeway segment between them.

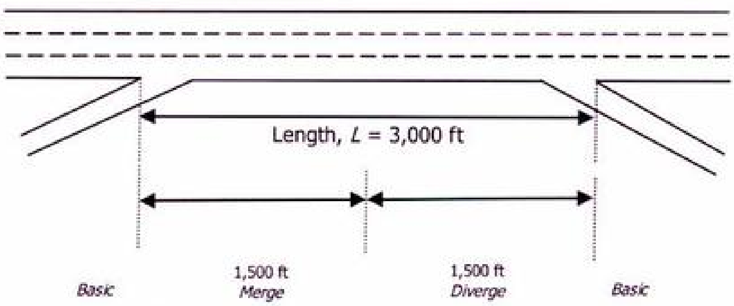
In Exhibit 10-11(b), the two ramps are 3,000 ft apart. The two 1,500-ft ramp influence areas define the entire length. Therefore, there is no basic freeway segment between the merge and diverge segments.

In Exhibit 10-11(c), the situation is more complicated. With only 2,000 ft between the ramps, the merge and diverge influence areas overlap for a distance of 1,000 ft.

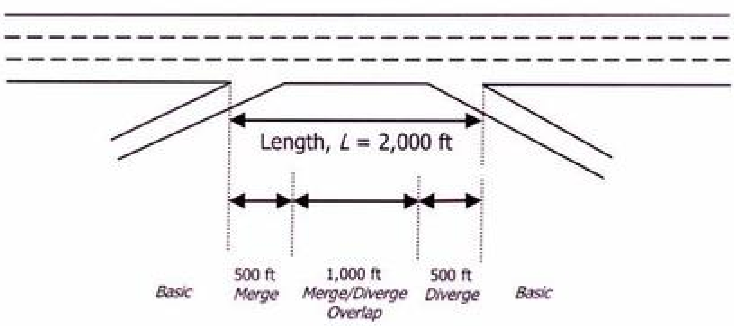
Exhibit 10-11
Defining Analysis Segments for a Ramp Configuration



(a) Section Length Between Ramps = 4,000 ft



(b) Section Length Between Ramps = 3,000 ft



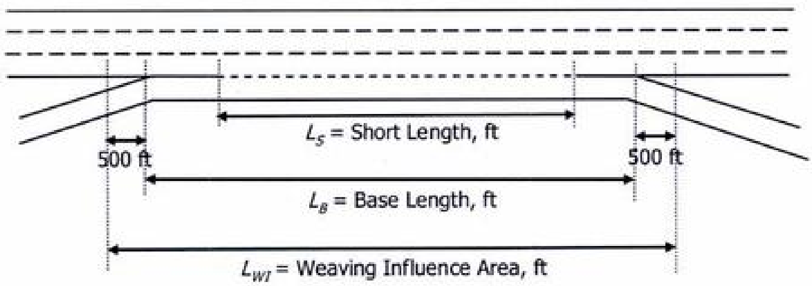
(c) Section Length Between Ramps = 2,000 ft

Chapter 14 covers this situation. Where ramp influence areas overlap, the analysis is conducted for each ramp separately. The analysis producing the worse LOS (or service measure value if the LOS is equivalent) is used to define operations in the overlap area.

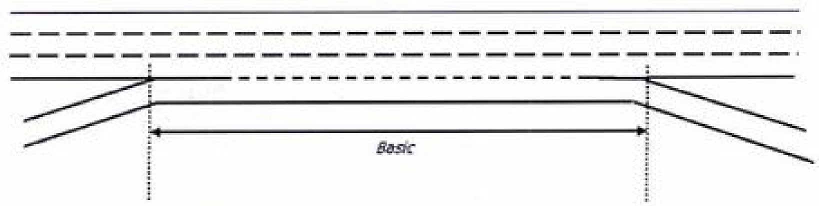
The facility methodology goes through the logic of distances and segment definitions to convert section boundaries to segment boundaries for analysis. If the distance between an on-ramp and an off-ramp is less than the full influence area of 1,500 ft, the worst case is applied to the distance between the ramps, while basic segment criteria are applied to segments upstream of the on-ramp and downstream of the off-ramp.

A similar situation can arise where weaving configurations exist. Exhibit 10-12 illustrates a weaving configuration within a defined freeway facility. In this case, the distance between the merge and diverge ends of the configuration must be compared with the maximum length of a weaving segment L_{wMAX}. If the distance between the merge and diverge points is less than or equal to L_{wMAX}, the entire segment is analyzed as a weaving segment, as shown in Exhibit 10-12(a).

Exhibit 10-12
Defining Analysis Segments for a Weaving Configuration



(a) Case I: $L_B \leq L_{wMAX}$ (weaving segment exists)



(b) Case II: $L_B > L_{wMAX}$ (analyze as basic segment)

Three lengths are involved in analyzing a weaving segment:

- The *short length* of the segment, defined as the distance over which lane changing is not prohibited or dissuaded by markings (L_S);
- The *base length* of the segment, measured from the points where the edges of the travel lanes of the merging and diverging roadways converge (L_B); and
- The *influence area* of the weaving segment (L_{WI}), which includes 500 ft upstream and downstream of L_B.

The influence area is the length that is used in all the predictive models for weaving segment analysis. However, the results of these models apply to a distance of L_B + 500 ft upstream to L_B + 500 ft downstream. For further discussion of the various lengths applied to weaving segments, consult Chapter 13.

If L_S is greater than L_{wMAX}, the merge and diverge segments are too far apart to form a weaving segment. As shown in Exhibit 10-12(b), the segment is treated as a basic freeway segment.

In the Chapter 13 weaving methodology, the value of L_{wMAX} depends on a number of considerations, including the split of component flows, demand flows, and other traffic factors. A weaving configuration could therefore qualify as a weaving segment in some analysis periods and as a separate merge, diverge, or basic segment in others.

In segmenting the freeway facility for analysis, merge, diverge, and weaving segments are identified as illustrated in Exhibit 10-11 and Exhibit 10-12. All segments not qualifying as merge, diverge, or weaving segments are basic freeway segments.

However, a long basic freeway section may have to be divided into multiple segments. This situation occurs when there is a sharp break in terrain within the section. For example, a 5-mi section may have a constant demand and a constant number of lanes. If there is a 2-mi level terrain portion followed by a 4% grade that is 3 mi long, the level terrain portion and the specific grade portion would be established as two separate consecutive basic freeway segments.

Step A-3: Input Data

Demand, geometry, and other data must be specified. Since the methodology builds on segment analysis, all data for each segment and each time period must be provided, as indicated in Chapters 12–14.

Demand

Demand flow rates must be specified for each segment and time period. Because analysis of multiple time periods is based on consecutive 15-min periods, the demand flow rates for each period must be provided. This condition is in addition to the requirements for isolated segment analyses.

Demand flow rates must be specified for the entering freeway mainline flow and for each on-ramp and off-ramp within the defined facility. The following information is needed for each time period to determine the demand flow rate:

- Demand flow rate (veh/h),
- Percent single-unit trucks and buses, and
- Percent tractor-trailer trucks.

For weaving segments, demand flow rates must be identified by component movement: freeway to freeway, ramp to freeway, freeway to ramp, and ramp to ramp. Where this level of detail is not available, the following procedure may be used to estimate the component flows. It is less desirable, however, since weaving segment performance is sensitive to the split of demand flows.

- *Ramp-weave segments:* Assume that the ramp-to-ramp flow is 0. The ramp-to-freeway flow is then equal to the on-ramp flow; the freeway-to-ramp flow is then equal to the off-ramp flow.
- *Major weave segments:* On-ramp flow is apportioned to the two exit legs (freeway and ramp) in the same proportion as the total flow on the exit legs (freeway and ramp).

Geometry

All geometric features for each segment of the facility must be specified, including the following:

- Number of lanes;
- Average lane width;
- Right-side lateral clearance;
- Terrain;
- Free-flow speed; and
- Location of merge, diverge, and weaving segments, with all internal geometry specified, including the number of lanes on ramps and at ramp–freeway junctions or within weaving segments, lane widths, existence and length of acceleration or deceleration lanes, distances between merge and diverge points, and the details of lane configuration where relevant.

Geometry does not change by time period, so this information is given only once, regardless of the number of time periods under study.

Effects of work zones, weather, and incidents can also be included in the analysis. Section 4 provides an extension of the method for work zone analysis. Chapter 11 provides default adjustment factors for weather and incident effects that can be used to calibrate the procedure in Step A-8.

Step A-4: Balance Demands

Traffic counts taken at each entrance to and exit from the defined freeway facility (including the mainline entrance and mainline exit) for each time interval serve as inputs to the methodology. While entrance counts are considered to represent the current entrance demands for the freeway facility (provided there is no queue on the freeway entrance), the exit counts may not represent the current exit demands for the freeway facility because of congestion within the defined facility.

For planning applications, estimated traffic demands at each entrance to and exit from the freeway facility for each time interval serve as inputs to the methodology. The sum of the input demands must equal the sum of the output demands in every time interval.

Once the entrance and exit demands are calculated, the demands for each cell in the time–space domain can be estimated for every time period. The segment demands can be thought of as filtering across the time–space domain and filling each cell of the time–space matrix.

Estimates of demand are needed when the methodology is applied by using actual freeway counts. If demand flows are known or can be projected, they are used directly without modification.

The methodology includes a demand estimation model that converts the input set of freeway exit 15-min counts to a set of vehicle flows that desire to exit the freeway in a given 15-min period. This demand may not be the same as the 15-min exit count because of upstream congestion within the defined freeway facility.

The procedure sums the freeway entrance demands along the entire directional freeway facility, including the entering mainline segment, and compares this sum with the sum of freeway exit counts along the directional freeway facility, including the departing mainline segment. This procedure is repeated for each time interval. When sensor data are used to populate the inputs for this procedure, the total entering and exiting demands in a time period may not be the same if there is congestion internal to the facility. The ratio of the total facility entrance counts to total facility exit counts is called the *time interval scale factor* and should approach 1.00 when the freeway exit counts are, in fact, freeway exit demands.

Scale factors greater than 1.00 indicate increasing levels of congestion within the freeway facility, with exit counts underestimating the actual freeway exit demands. To provide an estimate of freeway exit demand, each freeway exit count is multiplied by the time interval scale factor.

Equation 10-2 and Equation 10-3 summarize this process:

$$f_{TISi} = \frac{\sum_j V_{ON15ij}}{\sum_j V_{OFF15ij}}$$

Equation 10-2

$$V_{dOFF15ij} = V_{OFF15ij} \times f_{TISi}$$

Equation 10-3

where

f_{TISi} = time interval scale factor for time period i,

V_{ON15ij} = 15-min entering count for time period i and entering location j (veh),

$V_{OFF15ij}$ = 15-min exit count for time period i and exiting location j (veh), and

$V_{dOFF15ij}$ = adjusted 15-min exit demand for time period i and exiting location j (veh).

Once the entrance and exit demands are determined, the traffic demands for each section and each time period can be calculated. On the time–space domain, section demands can be viewed as projecting horizontally across Exhibit 10-10, with each cell containing an estimate of its 15-min demand.

Step A-5: Code Base Facility

This is the first step requiring the use of a computational engine or software. While individual time periods with undersaturated operations can be evaluated manually with this chapter's procedure, computations over multiple analysis periods and computations involving oversaturated segments and time periods require the use of a computational engine. A computational engine is available as part of Volume 4 of the HCM for research purposes. Commercial software packages that implement the method are also available.

Data input needs for the engine or other tools include all items collected or estimated in the previous steps. Data generally need to be entered for each segment and each time period, which makes this one of the most time-consuming steps in the analysis.

Step A-6: Identify Global Parameters

This step defines global (facilitywide) parameters that are needed for computation and calibration. While most inputs to the methodology can change at the segment and time period level, two global parameters are used across the entire spatial and temporal domains:

- *Jam density*, which is defined as the maximum achievable density in a segment under congested flow conditions, equivalent to a theoretical flow rate and segment speed of zero. The jam density affects the oversaturated speed–flow–density relationship used for calculations. The default value for jam density is 190 pc/mi/ln.
- *Queue discharge capacity drop,* which is defined as a percent reduction in the prebreakdown capacity following breakdown at an active bottleneck. The postbreakdown flow rate or queue discharge rate is defined as the average flow rate during oversaturated conditions (i.e., during the time interval after breakdown and before recovery). This factor directly affects

the throughput at a bottleneck and therefore the overall facility performance. The default value for the queue discharge capacity drop is 7%, on the basis of research (*7*).

Use of these parameters in the oversaturated flow portion of the methodology and their expected effects on calibration and validation are described in Chapter 25, Freeway Facilities: Supplemental.

Step A-7: Compute Segment Capacities

Segment capacity estimates are determined by the methodologies of Chapter 12 for basic freeway segments, Chapter 13 for weaving segments, and Chapter 14 for merge and diverge segments. All estimates of segment capacity should be carefully reviewed and compared with local knowledge and available traffic information for the study site, particularly where there are known bottlenecks.

On-ramp and off-ramp roadway capacities are also determined in this step with the Chapter 14 methodology. On-ramp demands may exceed on-ramp capacities, which would limit the traffic demand entering the facility. Off-ramp demands may exceed off-ramp capacities, which would cause congestion on the freeway, although that impact is not accounted for in this methodology.

All capacity results are stated in vehicles per hour under prevailing roadway and traffic conditions.

The effect of a predetermined ramp-metering plan can be evaluated in this methodology by overriding the computed ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to reflect the specified ramp-metering rate. This approach not only allows for evaluating a prescribed ramp-metering plan but also permits the user to improve the ramp-metering plan through experimentation. The analysis can further estimate the extent of on-ramp queuing, but the same queue density as the mainline queue is assumed.

Freeway design improvements can be evaluated with this methodology by modifying the design features of any portion of the freeway facility. For example, the effects of adding auxiliary lanes at critical locations and full lanes over multiple segments can be assessed.

Step A-8: Calibrate with Adjustment Factors

Segment capacities can be affected by a number of conditions, some of which may not normally be accounted for in the segment methodologies of Chapters 12–14. These reductions include the effects of adverse weather conditions, other environmental factors, driver population, and incidents. Adjustments for work zones and lane closures for construction or major maintenance operations are discussed in Section 4.

This step allows the user to adjust demands, capacities, and free-flow speeds for the purpose of calibration or to reflect the impacts of weather, incidents, and work zones. The demand adjustment factor DAF_{cal}, capacity adjustment factor CAF_{cal}, and speed adjustment factor SAF_{cal} can be modified for each segment and each time period. The adjustment factors are used as multipliers for the base demand, capacity, and free-flow speeds input into the methodology.

It is strongly recommended that *the base run not include any adjustments*, with the three adjustment factors above being used as calibration tools in one or more subsequent iterations with the intent of matching field data. CAF and SAF values should always be equal to or less than 1.0, since they are intended to adjust the base values downwards. If needed, a higher base value can be used and then calibrated downward with the CAF and SAF factors. DAFs are primarily used in the context of a freeway reliability analysis, as discussed in Chapter 11.

An adjusted free-flow speed FFS_{adj} is calculated by multiplying the FFS by a SAF_{cal} as shown in Equation 10-4:

$$FFS_{adj} = FFS \times SAF_{cal}$$

Equation 10-4

An adjusted capacity c_{adj} is calculated by multiplying the base capacity c by a CAF_{cal} as shown in Equation 10-5:

$$c_{adj} = c \times CAF_{cal}$$

Equation 10-5

An adjusted demand input volume v_{adj} is calculated by multiplying the base demand volume v by a DAF_{cal} as shown in Equation 10-6:

$$v_{adj} = v \times DAF_{cal}$$

Equation 10-6

At lane drops, permanent reductions in capacity occur. These effects are included in the core methodology, which determines segment capacity on the basis of the number of lanes in the segment and other prevailing conditions. However, the method does not account for frictional effects at lane drops, which may be needed to calibrate the facility operation properly.

Speed and capacity adjustment factors for weather, other environmental conditions, and incidents are found in Chapter 12, Basic Freeway and Multilane Highway Segments. Adjustments for driver population characteristics are discussed in Chapter 26; since no default values for driver population adjustments are presently available, these adjustments need to be estimated locally. The application of these adjustment factors to different segment types is described in Chapters 12, 13, and 14 as applicable.

Step A-9: Managed Lane Cross-Weave Adjustment

This step is only required for facilities with managed lanes. It implements a friction factor for traffic from a general purpose on-ramp that weaves across the general purpose lanes to get to a managed lane access point (or vice versa). The cross-weave adjustment factor is conceptually similar to the CAF used in Step A-8 and is discussed in detail in Section 4.

Step A-10: Compute Demand-to-Capacity Ratios

Each cell of the time–space domain now contains an estimate of demand and capacity. A demand-to-capacity ratio can be calculated for each cell. The cell values must be carefully reviewed to determine whether all boundary cells have v_d/c ratios of 1.00 or less and to determine whether any cells in the interior of the time–space domain have v_d/c values greater than 1.00.

If any boundary cells have a v_d/c ratio greater than 1.00, further analysis may be significantly flawed:

1. If any cell in the first time interval has a v_d/c ratio greater than 1.00, there may have been oversaturated conditions in earlier time intervals without transfer of unsatisfied demand into the time–space domain of the analysis.
2. If any cell in the last time interval has a v_d/c ratio greater than 1.00, the analysis will be incomplete because the unsatisfied demand in the last time interval cannot be transferred to later time intervals.
3. If any cell in the last downstream segment has a v_d/c ratio greater than 1.00, there may be downstream bottlenecks that should be checked before proceeding with the analysis. If any cell in the first segment has a v_d/c ratio greater than 1.00, oversaturation will extend upstream of the defined freeway facility, but its effects will not be analyzed within the time–space domain.

These checks do not guarantee that the boundary cells will not show v_d/c ratios greater than 1.00 later in the analysis. If these initial checks reveal boundary cells with v_d/c ratios greater than 1.00, the time–space domain of the analysis should be adjusted to eliminate the problem.

As the analysis of the time–space domain proceeds, subsequent demand shifts may cause some boundary cell v_d/c ratios to exceed 1.00. In these cases, the problem should be reformulated or alternative tools applied. Most alternative tools will have the same problem if the boundary conditions experience congestion.

Another important check is to observe whether any cell in the interior of the time–space domain has a v_d/c ratio greater than 1.00. There are two possible outcomes:

1. If all cells have v_d/c ratios of 1.00 or less, the entire time–space domain contains undersaturated flow, and the analysis is greatly simplified.
2. If any cell in the time–space domain has a v_d/c ratio greater than 1.00, the time–space domain will contain both undersaturated and oversaturated cells. Analysis of oversaturated conditions is much more complex because of the interactions between freeway segments and the shifting of demand in both time and space.

If Case 1 exists, the analysis moves to Step A-11. If Case 2 exists, the analysis moves to Step A-12.

The v_d/c ratio for all on-ramps and off-ramps should also be examined. If an on-ramp demand exceeds the on-ramp capacity, the ramp demand flow rates should be adjusted to reflect capacity. Off-ramps generally fail because of deficiencies at the ramp–street junction. They may be analyzed by procedures in Chapters 19–22, depending on the type of traffic control used at the ramp–street junction. These checks are done manually, and inputs to this methodology must be revised accordingly.

Steps A-11 and A-12: Compute Undersaturated and Oversaturated Performance Measures

The analysis begins in the first cell in the upper-left corner of the time–space domain (the first segment in the first time interval) and continues downstream along the freeway facility for each segment in the first time interval. The analysis then returns to the first upstream segment in the second time interval and continues downstream along the freeway for each segment in the second time interval. This process continues until all cells in the time–space domain have been analyzed (refer back to Exhibit 10-10 for an illustrative example).

As each cell is analyzed in turn, its v_d/c ratio is checked. If the v_d/c ratio is 1.00 or less, the cell is not a bottleneck and is able to handle all traffic demand that wishes to enter. The process is continued in the order noted in the previous paragraph until a cell with a v_d/c ratio greater than 1.00 is encountered. Such a cell is labeled as a bottleneck. Because the bottleneck cannot handle a flow greater than its capacity, the following impacts will occur:

1. The v_a/c ratio of the bottleneck cell will be exactly 1.00, since the cell processes a flow rate equal to its capacity.
2. Flow rates for all cells downstream of the bottleneck must be adjusted downward to reflect the fact that not all the demand flow at the bottleneck is released. Downstream cells are subject to demand starvation due to the bottleneck metering effect.
3. The unsatisfied demand at the bottleneck cell must be stored in the upstream segments. Flow conditions and performance measures in these upstream cells are affected. Shock wave analysis is applied to estimate these impacts.
4. The unsatisfied demand stored upstream of the bottleneck cell must be transferred to the next time interval. The transfer is accomplished by adding the unsatisfied demand by desired destination to the origin–destination table of the next time interval.

This four-step process is implemented for each bottleneck encountered, following the specified sequence of cell analysis. If no bottlenecks are identified, the entire domain is undersaturated, and the sequence of steps for oversaturated conditions is not applied.

If a bottleneck is severe, the storage of unsatisfied demand may extend beyond the upstream boundary of the freeway facility or beyond the last time interval of the time–space domain. In such cases, the analysis will be flawed, and the time–space domain should be reconstituted.

After all demand shifts (in the case of one or more oversaturated cells) are estimated, each cell is analyzed by the methodologies of Chapters 11, 12, and 13. Facility service and performance measures may then be estimated.

Step A-11: Undersaturated Conditions

For undersaturated conditions, the process is straightforward. Because there are no cells with v_d/c ratios greater than 1.00, the flow rate in each cell v_a is equal to the demand flow rate v_d. Each segment analysis using the methodologies of Chapters 12–14 will result in estimating a density D and a space mean speed S.

When the analysis moves from isolated segments to a facility, additional constraints may be necessary. A maximum-achievable-speed constraint is imposed to limit the prediction of speeds in segments downstream of a segment experiencing low speeds. This constraint prevents large speed fluctuations from segment to segment when the segment methodologies are directly applied. This process results in some changes in the speeds and densities predicted by the segment methodologies.

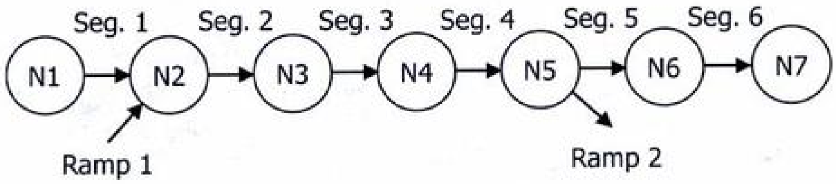
For each time interval, Equation 10-1 is used to estimate the average density for the defined freeway facility. This result is compared with the criteria of Exhibit 10-6 to determine the facility LOS for the time period. Each time period will have a separate LOS. Although LOS is not averaged over time intervals, if desired, density can be averaged over time intervals.

Step A-12: Oversaturated Conditions

Once oversaturation is encountered, the methodology changes its temporal and spatial units of analysis. The spatial units become nodes and segments, and the temporal unit moves from a time interval of 15 min to smaller time periods, as recommended in Chapter 25, Freeway Facilities: Supplemental.

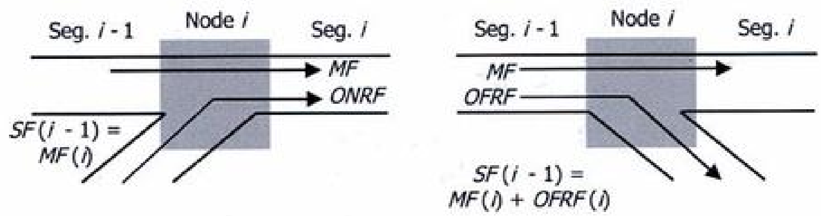
Exhibit 10-13 illustrates the node–segment concept. A node is defined as the junction of two segments. Since there is a node at the beginning and end of the freeway facility, there will always be one more node than the number of segments on the facility.

Exhibit 10-13
Node–Segment Representation of a Freeway Facility



The numbering of nodes and segments begins at the upstream end of the defined freeway facility and moves to the downstream end. The segment upstream of node i is numbered $i - 1$, and the downstream segment is numbered i, as shown in Exhibit 10-14.

Exhibit 10-14
Mainline and Segment Flow at On- and Off-Ramps



Note: *SF* = segment flow, *MF* = mainline flow, *ONRF* = on-ramp flow, and *OFRF* = off-ramp flow.

The oversaturated analysis moves from the first node to each downstream node for a time step. After the analysis for the first time step is complete, the same nodal analysis is performed for each subsequent time step.

When oversaturated conditions exist, many flow variables must be adjusted to reflect the upstream and downstream effects of bottlenecks. These adjustments are explained in general terms in the sections that follow and are fully detailed in Chapter 25.

Flow Fundamentals

As noted previously, segment flow rates must be calculated for each time step. They are used to estimate the number of vehicles on each segment at the end of every time step. The number of vehicles on each segment is used to track queue accumulation and discharge and to estimate the average segment density.

The conversion from standard 15-min time intervals to time steps (of lesser duration) occurs during the first oversaturated interval. Time steps are then used until the analysis is complete. This transition to time steps is critical because, at certain points in the methodology, future performance is estimated from the past performance of an individual variable. The use of time steps also allows for a more accurate estimation of queues.

Service and other performance measures for oversaturated conditions use a simplified, linear flow–density relationship, as detailed in Chapter 25.

Segment Initialization

To estimate the number of vehicles on each segment for each time step under oversaturated conditions, the process must begin with the appropriate number of vehicles in each segment. Determining this number is referred to as *segment initialization*.

A simplified queuing analysis is initially performed to account for the effects of upstream bottlenecks. The bottlenecks limit the number of vehicles that can proceed downstream.

To obtain the proper number of vehicles on a given segment, an *expected demand* is calculated that includes the effects of all upstream segments. The expected demand represents the flow that would arrive at each segment if all queues were stacked vertically (i.e., as if the queues had no upstream impacts). For all segments upstream of a bottleneck, the expected demand will equal the actual demand.

For the bottleneck segment and all further downstream segments, a capacity restraint is applied at the bottleneck when expected demand is computed. From the expected segment demand, the background density can be obtained for each segment by using the appropriate estimation algorithms from Chapters 12–14.

Mainline Flow Calculation

Flows analyzed in oversaturated conditions are calculated for every time step and are expressed in vehicles per time step. They are analyzed separately on the basis of the origin and destination of the flow across the node. The following flows are defined:

1. The flow from the mainline upstream segment $i - 1$ to the mainline downstream segment i is the mainline flow MF.

2. The flow from the mainline to an off-ramp is the off-ramp flow *OFRF*.
3. The flow from an on-ramp to the mainline is the on-ramp flow *ONRF*.

Each of these flows was illustrated in Exhibit 10-14.

Mainline Input

The mainline input is the number of vehicles that wish to travel through a node during the time step. The calculation includes the effects of bottlenecks upstream of the subject node. These effects include the metering of traffic during queue accumulation and the presence of additional vehicles during queue discharge.

The mainline input is calculated by taking the number of vehicles entering the node upstream of the analysis node, adding on-ramp flows or subtracting off-ramp flows, and adding the number of unserved vehicles on the upstream segment. The result is the maximum number of vehicles that desire to enter a node during a time step.

Mainline Output

The mainline output is the maximum number of vehicles that can exit a node, constrained by downstream bottlenecks or by merging traffic. Different constraints on the output of a node result in three different types of mainline outputs (MO1, MO2, and MO3).

- *Mainline output from ramps (MO1):* MO1 is the constraint caused by the flow of vehicles from an on-ramp. The capacity of an on-ramp flow is shared by two competing flows: flow from the on-ramp and flow from the mainline. The total flow that can pass the node is estimated as the minimum of the segment *i* capacity and the mainline outputs (MO2 and MO3) calculated in the preceding time step.
- *Mainline output from segment storage (MO2):* The output of mainline flow through a node is also constrained by the growth of queues on the downstream segment. The presence of a queue limits the flow into the segment once the queue reaches its upstream end. The queue position is calculated by shock wave analysis. The MO2 limitation is determined first by calculating the maximum number of vehicles allowed on a segment at a given queue density. The maximum flow that can enter a queued segment is the number of vehicles leaving the segment plus the difference between the maximum number of vehicles allowed on a segment and the number of vehicles already on the segment. The queue density is determined from the linear congested portion of the density–flow relationship shown in Chapter 25.
- *Mainline output from front-clearing queue (MO3):* The final limitation on exiting mainline flows at a node is caused by front-clearing downstream queues. These queues typically occur when temporary incidents clear. Two conditions must be satisfied: (*a*) the segment capacity (minus the on-ramp demand if present) for the current time interval must be greater than the segment capacity (minus on-ramp demand) in the preceding time interval, and (*b*) the segment capacity minus the ramp demand for

the current time interval must be greater than the segment demand in the same time interval. Front-clearing queues do not affect the segment throughput (which is limited by queue throughput) until the recovery wave has reached the upstream end of the segment. The shock wave speed is estimated from the slope of the line connecting the bottleneck throughput and the segment capacity points.

Mainline Flow

The mainline flow across node *i* is the minimum of the following variables:

- Node *i* mainline input,
- Node *i* MO2,
- Node *i* MO3,
- Segment $i - 1$ capacity, and
- Segment *i* capacity.

Determining On-Ramp Flow

The on-ramp flow is the minimum of the on-ramp input and output. Ramp input in a time step is the ramp demand plus any unserved ramp vehicles from a previous time step.

On-ramp output is limited by the ramp roadway capacity and the ramp-metering rate. It is also affected by the volumes on the mainline segments. The latter is a complex process that depends on the various flow combinations on the segment, the segment capacity, and the ramp roadway volumes. Details of the calculations are presented in Chapter 25.

Determining Off-Ramp Flow

The off-ramp flow is determined by calculating a diverge percentage on the basis of the segment and off-ramp demands. The diverge percentage varies only by time interval and remains constant for vehicles that are associated with a particular time interval. If there is an upstream queue, traffic to this off-ramp may be metered. This will cause a decrease in the off-ramp flow. When vehicles that were metered arrive in the next time interval, they use the diverge percentage associated with the preceding time interval. This methodology ensures that all off-ramp vehicles prevented from exiting during the presence of a bottleneck are appropriately discharged in later time intervals.

Determining Segment Flow

Segment flow is the number of vehicles that flow out of a segment during the current time step. These vehicles enter the current segment either to the mainline or to an off-ramp at the current node, as shown in Exhibit 10-13. The number of vehicles on each segment in the current time step is calculated on the basis of

- The number of vehicles that were in the segment in the previous time step,
- The number of vehicles that entered the segment in the current time step, and
- The number of vehicles that can leave the segment in the current time step.

Because the number of vehicles that leave a segment must be known, the number of vehicles on the current segment cannot be determined until the upstream segment is analyzed.

The number of unserved vehicles stored on a segment is calculated as the difference between the number of vehicles on the segment and the number of vehicles that would be on the segment at the background density.

Determining Segment Service Measures

In the last time step of a time interval, the segment flows in each time step are averaged over the time interval, and the service measures for each segment are calculated. If there were no queues on a particular segment during the entire time interval, the performance measures are calculated from Chapters 12, 13, and 14 as appropriate.

If there was a queue on the current segment during the time interval, the performance measures are calculated in four steps:

1. The average number of vehicles over a time interval is calculated for each segment.
2. The average segment density is calculated by taking the average number of vehicles in all time steps (in the time interval) and dividing it by the segment length.
3. The average speed on the current segment during the current time interval is calculated as the ratio of segment flow to density.
4. The final segment performance measure is the length of the queue at the end of the time interval (if one exists), which is calculated by using shock wave theory.

On-ramp queue lengths can also be calculated. A queue will form on the on-ramp roadway only if the flow is limited by a meter or by freeway traffic in the gore area. If the flow is limited by the ramp roadway capacity, unserved vehicles will be stored on a facility upstream of the ramp roadway, most likely a surface street. The methodology does not account for this delay. If the queue is on a ramp roadway, its length is calculated by using the difference in background and queue densities.

Step A-13: Apply Managed Lane Adjacent Friction Factor

This step adjusts the performance of (undersaturated) managed lanes when the adjacent general purpose lanes operate with a density greater than 35 pc/mi/ln, depending on the separation type between the two lane groups (i.e., paint, buffer, barrier). This step only applies to facilities with managed lanes and is discussed in more detail in Section 4.

Step A-14: Compute Lane Group Performance

This step computes the performance measure for the length of the facility for each lane group separately. This step only applies to facilities with managed lanes and is discussed in more detail in Section 4.

Step A-15: Compute Freeway Facility Performance Measures by Time Interval

The previously discussed traffic performance measures can be aggregated over the length of the defined freeway facility for each analysis period. Aggregations over the entire time–space domain of the analysis are also mathematically possible, although LOS is defined only for each 15-min analysis period.

The performance measures include the computation of queue spillback under oversaturated conditions. All congestion should be fully contained within the specified time–space domain. If congestion remains at the end of the last time interval or if queues spill back beyond the first segment at any time in the analysis, the analysis returns to Step A-5 and the time–space domain is expanded accordingly.

Step A-16: Aggregate to Section Level and Validate Against Field Data

In this step, the aggregated methodology results at the section level are compared with field data or results from another model. Additional details on criteria for calibration and validation of the facility are provided in Chapter 25, Freeway Facilities: Supplemental. If an acceptable match is not obtained, the analysis returns to Step A-6 and follows the steps for calibration adjustments.

Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility

This final step of the core methodology estimates the LOS for each segment, lane group, and the overall facility for each time period. Freeway facility LOS is defined for each time interval included in the analysis. An average density for each time interval, weighted by length of segments and numbers of lanes in segments, is calculated by using Equation 10-1 and is compared against the criteria of Exhibit 10-6.

Step A-17 concludes the core freeway facility methodology for the analysis of a single study period analysis. However, the analyst may choose to continue to perform a reliability analysis or evaluation of ATDM strategies as described in Chapter 11, Freeway Reliability Analysis.

capacities over time, but no conclusive evidence in this regard was found in the research.

The lane closure configuration in a work zone is expressed as *the ratio of the number of original lanes to the number of lanes present in the work zone*. For instance, a 3-to-1 lane closure configuration means that three lanes are normally available, but that two lanes were closed during construction and only one lane was open. Research indicates that this ratio is effective in showing the influences of different lane configurations on speed or capacity.

This ratio cannot distinguish a 4-to-2 lane closure configuration from a 2-to-1 configuration, since both yield a ratio of 0.5. Field observations (5) and citations in the literature (4) both suggest that the per lane capacity of a 2-to-1 lane closure is significantly less than that of a 4-to-2 closure, due to fewer open lanes being available. The lane closure severity index (LCSI) distinguishes such lane closure configurations. Equation 10-7 shows how the LCSI is calculated:

Equation 10-7

$$LCSI = \frac{1}{OR \times N_o}$$

where

$LCSI$ = lane closure severity index (decimal);

OR = open ratio, the ratio of the number of open lanes during road work to the total (or normal) number of lanes (decimal); and

N_o = number of open lanes in the work zone (ln).

The LCSI clearly gives a unique value for different lane closure configurations, where higher values generally correspond to a more severe lane closure scenario. This is illustrated in Exhibit 10-15.

Exhibit 10-15
Lane Closure Severity Index Values for Different Lane Closure Configurations

Number of Total Lane(s)	Number of Open Lane(s)	Open Ratio	LCSI
3	3	1.00	0.33
2	2	1.00	0.50
4	3	0.75	0.44
3	2	0.67	0.75
4	2	0.50	1.00
2	1	0.50	2.00
3	1	0.33	3.00
4	1	0.25	4.00

Note: LCSI = lane closure severity index.

In interpreting Exhibit 10-15, it is noted that not all work zones are associated with lane closure effects. For example, work zones may be limited to shoulder work only or may feature a lane shift or crossover. This chapter's methodology also applies to work zones without lane closures. In the exhibit, a "2-to-2 work zone" can refer to shoulder closures or crossovers that do not affect the overall number of travel lanes.

Adjustments to the Core Methodology

Work Zone Capacity and Queue Discharge Rate Model

Freeway work zone capacity corresponds to the maximum sustainable flow rate immediately preceding a breakdown. However, measuring the prebreakdown value in work zones is often not feasible. On the other hand, queue discharge flow rates can easily be measured by using video cameras or other data collection tools. Therefore, to arrive at an estimate of prebreakdown work zone capacity, models to predict queue discharge rate as a function of work zone configurations and other prevailing conditions are presented. The queue discharge rate is then converted back to the corresponding prebreakdown flow rate by using a conversion ratio.

The work zone queue discharge rate is defined as follows:

The average flow rate immediately downstream of an active bottleneck (following breakdown) measured over a 15-min sampling interval while there is active queuing upstream of the bottleneck.

Equation 10-8 gives a predictive model for freeway work zone queue discharge rate as a function of the work zone configuration and other prevailing conditions:

$$QDR_{wz} = 2{,}093 - 154 \times LCSI - 194 \times f_{Br} - 179 \times f_{AT} + 9 \times f_{LAT} - 59 \times f_{DN}$$

Equation 10-8

where QDR_{wz} is the average 15-min queue discharge rate (pc/h/ln) at the work zone bottleneck.

As expected, the work zone queue discharge rate is lower at higher LCSI values, when soft barriers are present, in rural areas, with smaller lateral clearances, and at night.

The prebreakdown capacity for work zones can be estimated from the queue discharge flow rate, which is expected to be lower than the prebreakdown flow rate. Equation 10-9 is used to determine the prebreakdown capacity:

$$c_{wz} = \frac{QDR_{wz}}{100 - \alpha_{wz}} \times 100$$

Equation 10-9

where c_{wz} is the work zone capacity (prebreakdown flow rate) (pc/h/ln), α_{wz} is the percentage drop in prebreakdown capacity at the work zone due to queuing conditions (%), and QDR_{wz} is as defined above.

Research shows an average queue discharge drop of 7% in non–work zone conditions (*7*) and an average value of 13.4% in freeway work zones (*4*). The underlying research measured prebreakdown capacities as well as queue discharge rates to estimate the magnitude of α_{wz}. When there is little local information available on α_{wz}, these values can be used as defaults.

The calculated work zone capacity should not be greater than the non–work zone capacity, and the result of Equation 10-9 should be capped as necessary.

Work Zone Free-Flow Speed Model

A model for work zone free-flow speed has been developed through work zone observations during low-flow conditions. The model should only be used if no local estimates of FFS are available.

Equation 10-10 predicts FFS in freeway work zones on the basis of work zone configurations and other prevailing conditions:

Equation 10-10

$$FFS_{wz} = 9.95 + 33.49 \times f_{Sr} + 0.53 \times SL_{wz} - 5.60 \times LCSI - 3.84 \times f_{Br} - 1.71 \times f_{DN} - 8.7 \times TRD$$

where FFS_{wz} is the work zone free-flow speed (mi/h) and all other variables are as defined previously.

The work zone FFS decreases as the LCSI increases, when soft barriers are used, at night, and as the ramp density increases. Higher work zone speed limits and higher speed ratios result in higher work zone FFS.

The calculated work zone FFS should not be greater than the non–work zone FFS, and the result of Equation 10-10 should be capped as needed.

Work Zone Speed–Flow Model

Changes in work zone prebreakdown capacity and work zone FFS influence the overall shape of the speed–flow model in the freeway segments affected by the work zone. Work zone FFS is determined with Equation 10-10, while work zone capacity is determined with Equation 10-8 and Equation 10-9.

Adjustment factors for capacity and FFS are used to reflect the effect of the work zone on speeds and flows. Equation 10-11 is used to determine the work zone capacity adjustment factor.

Equation 10-11

$$CAF_{wz} = \frac{c_{wz}}{c}$$

where

CAF_{wz} = capacity adjustment factor for a work zone (decimal),

c = basic freeway segment capacity in non–work zone conditions (pc/h/ln), and

c_{wz} = work zone capacity (prebreakdown flow rate) (pc/h/ln).

Similarly, Equation 10-12 is used to determine the speed adjustment factor for work zone conditions:

Equation 10-12

$$SAF_{wz} = \frac{FFS_{wz}}{FFS}$$

where

SAF_{wz} = free-flow speed adjustment factor for work zone (decimal),

FFS = freeway free-flow speed in non–work zone conditions (mi/h), and

FFS_{wz} = work zone free-flow speed (mi/h).

The calculated capacity and speed adjustment factors are inputs to the generic basic segment speed–flow relationship described in Chapter 12, Basic Freeway and Multilane Highway Segments (see Exhibit 12-6).

CAFs and SAFs for work zones should never be greater than 1.0, and the results of Equation 10-11 and Equation 10-12 should be capped at 1.0 accordingly.

Adjustments for Other Segment Types

The queue discharge rate model described above applies to basic freeway segments. Its estimates should be adjusted further for special freeway work zone configurations, such as merge segments, diverge segments, weaving segments, and work zones with directional crossovers. The relationships presented in this section were derived from field-calibrated microsimulation models for the special work zone configurations.

No data were available for the impacts of these configurations on FFS, so these estimates should be used only when local data are not available. One exception is the FFS for a directional crossover, which should be estimated on the basis of the crossover's geometric design and is subsequently used as an input to the queue discharge rate estimation.

Details on the adjustments for special work zone configurations are provided in Chapter 25, Freeway Facilities: Supplemental.

Special Work Zone Considerations

Other special considerations apply to work zones with small lateral clearances, significant heavy vehicle presence, or steep grades. These impacts are discussed below.

Minimum Lateral Distance

Observations have shown that work zones with minimum lateral clearances can have capacity and free-flow speeds well below the estimates given by the above models. One such example is shown in Exhibit 10-16. As seen in the exhibit, lateral clearances on both sides of the road are minimal and are constrained by concrete barriers. As a result, vehicles have limited ability to maneuver, which reduces capacity and FFS.

Exhibit 10-16
Example of Minimum Lateral Clearance in Work Zone

Note: I-5, Los Angeles, California.

Consequently, work zones with minimum lateral clearance on both sides are expected to have greatly reduced prebreakdown capacities, queue discharge flows, and free-flow speeds. Analysts should use caution in applying the average QDR and FFS models under these conditions.

Significant Heavy Vehicle Presence on Steep Grades

The model given previously for work zone queue discharge rate is in units of passenger cars and therefore incorporates the effects of terrain and heavy vehicle presence. Headways of heavy vehicles in freeway work zones are consistent with those on freeway segments without work zones; therefore, no additional work zone–specific heavy vehicle adjustment factors are provided.

However, special considerations apply when work zones provide only one open lane, since vehicles have no ability to pass slower heavy vehicles. On steep upgrades, heavy vehicles may slow to crawl speeds, as discussed in Chapter 12. In this case, the traffic following these heavy vehicles will also travel at crawl speed and the work zone capacity will be lower. An example of a freeway work zone with only one open lane, a high percentage of heavy vehicles, and a relatively long upgrade is shown in Exhibit 10-17.

Exhibit 10-17
Freeway Work Zone with One Open Lane, Trucks, and a Long Upgrade

Source: Nevada DOT.
Note: I-80, near Elko, Nevada.

MANAGED LANES ANALYSIS

This section provides a method for analyzing the operational performance of facilities with one or more managed lanes, as well as their interaction with the adjacent general purpose lanes. It does not evaluate the capacity of a dynamic managed lane, which is determined from the pricing algorithms. Similarly, it does not provide demand predictions or estimate changes in demand as a function of different pricing strategies. The methodology is largely based on the results from NCHRP Project 03-96 (*1*). Managed lanes may include HOV lanes, HOT lanes, or express toll lanes.

Four types of managed lane (ML) freeway segments are defined in Chapters 12 through 14: *ML merge and diverge segments, ML weaving segments, ML access segments,* and *basic freeway segments.*

The analysis procedures for general purpose lanes with adjacent managed lanes build on the core methodology's segment classification. In addition, the *lane group* concept is introduced to allow analysts to assign separate attributes to managed and general purpose lanes, while retaining a degree of interaction between the two facilities. The adjacent lane groups (one general purpose and one managed) are required to have the same segment length.

The research supporting this chapter found that the composition, FFS, capacity, and behavior characteristics of managed lane traffic streams are different from those of general purpose lanes. In addition, interaction effects between the two lane groups were observed, especially in cases where no physical barrier separated the managed and general purpose lanes.

Spatial and Temporal Limits

Similar to the freeway facility core methodology analysis, the managed lane methodology is limited to 15-min analysis periods as the smallest time unit. The spatial and temporal limits are consistent with the core methodology.

Limitations of the Methodology

The managed lane analysis methodology has the following limitations:

1. The methodology cannot address the interaction of merge and diverge maneuvers occurring at the start and end of the managed lane facility within the spatial limits of the analysis.
2. The impact of variations in the design of the start and end access points of the managed lane facilities and the operational impacts from variations in the design of the termini are not considered.
3. The methodology does not involve demand estimation, especially demand dynamics due to a pricing component that may be in effect on the managed lane facility. Demand is considered to be a time-dependent input to the method.
4. Managed and general purpose lanes must be jointly assigned in a feasible lane group. Adjacent managed lane and general purpose segments are required to have identical lengths and separation type. When a managed lane is added to an analysis, the general purpose lane segmentation may change.
5. Queue interactions between general purpose and managed lanes on the access segments are not explicitly considered in this methodology. However, the methodology will account for the delay caused by the presence of queues on access segments.
6. Multiple overlapping breakdowns or bottlenecks on either the general purpose or the managed lanes are not analyzed and cannot be fully evaluated by the managed lane methodology. Alternative tools may be

more appropriate for specific applications beyond the capabilities of the methodology.

7. Spatial, temporal, modal, and total demand responses to traffic management strategies are not automatically incorporated into the managed lane methodology. On viewing the facility traffic performance results, the analyst can modify the input demand manually to analyze the effect of user-demand responses and traffic growth. The accuracy of the results depends on the accuracy of the estimated user-demand responses.
8. The results should be viewed cautiously if the *d/c* ratio is greater than 1.00 for one or more freeway segments during the first or last analysis period or for the first freeway segment in any analysis period.
9. The method does not address conditions in which managed lane off-ramp capacity limitations result in queues that extend onto the managed lanes or affect the behavior of managed lane off-ramp vehicles.

In addition, all limitations of the core methodology apply equally to the managed lane extensions. Because this chapter's methodology incorporates the methodologies for basic, weaving, merging, and diverging freeway segments for both managed and general purpose lanes, the limitations of those procedures apply here.

Required Data and Sources

For a typical operational analysis, the analyst must specify demand volumes, roadway geometric information (including number of lanes, lane width, right-side lateral clearance, and total ramp density), percent heavy vehicles, peak hour factors, terrain, and capacity and speed calibration factors, similar to what is required for a general purpose freeway facility analysis. The only difference is that this information must be specified separately for the managed and general purpose lane groups. In addition, the type of separation provided between the managed and general purpose lanes must be specified.

Adjustments to the Base Methodology

Lane Group Concept

To capture the interaction effects between the managed and general purpose lanes while allowing for varying demand, capacity, and speed inputs, the concept of *lane groups* is introduced for freeway facilities with managed lanes. By adopting the lane group concept, an analyst can define separate attributes for parallel managed lane and general purpose facilities while retaining the ability to model the interaction between the two facilities.

Each segment of a freeway facility is represented as having either one or two lane groups, depending on whether a concurrent managed lane segment is present. Input variables such as geometric characteristics (e.g., number of lanes), traffic performance attributes (e.g., FFS, capacity), and traffic demands must be entered separately for each lane group. The methodology is then applied to assess the operational performance of each lane group, with consideration given to the empirically derived interaction effects between the two lane groups.

The following principles apply:

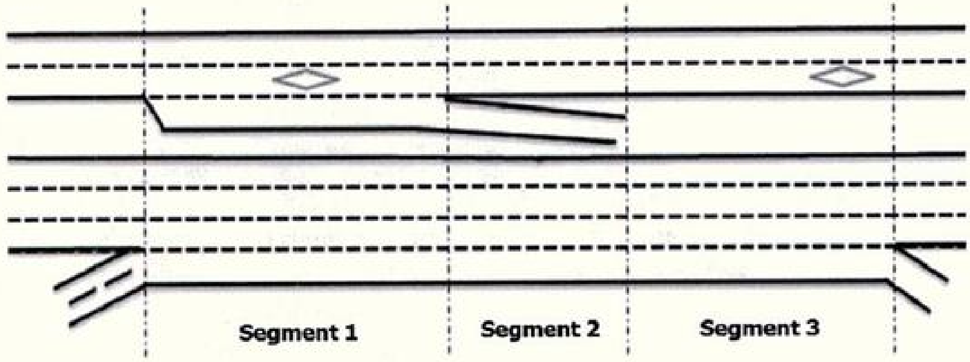
- A freeway general purpose segment with a parallel managed lane segment is considered as two adjacent lane groups.
- Adjacent lane groups (one general purpose and one managed lane segment) must have identical segment lengths.
- Adjacent lane groups can be of different segment types. For example, a basic managed lane segment may be concurrent with a general purpose diverge segment (see Exhibit 10-18 illustrating this case).
- Adjacent lane groups may have different geometric characteristics, including number of lanes, lane widths, and shoulder clearance.
- Adjacent lane groups may have unique operational attributes, including FFS, segment capacity, or various capacity- or speed-reducing factors.
- Adjacent lane groups may have unique traffic demand parameters, which are entered by the user and obtained through an external process. This chapter's operational methodology does not predict the split in demand between the managed and general purpose lanes.
- The operational performance of adjacent managed and general purpose lane groups is interdependent, in that congestion in one lane group may have a frictional effect on operations in the adjacent lane group. This frictional effect was empirically derived, can be user-calibrated, and is sensitive to the type of physical separation between lane groups (i.e., striping, buffer, barrier).

Oversaturated managed lane facilities are relatively rare in practice, since one of the underlying principles for managed lane operations (especially for tolled facilities) is to ensure that managed lane traffic density is below the critical density even in peak periods, which in turn guarantees satisfactory service to managed lane customers. However, congestion on managed and general purpose lanes can and should be considered by the method, because many facilities operate during peak periods, and especially in view of nonrecurring congestion effects (e.g., weather, incidents). Chapter 25 provides details on evaluating oversaturated managed and general purpose lanes.

Segmentation Considerations

To preserve the lane group concept, the segmentation is performed slightly differently from that for a freeway facility consisting only of general purpose lanes. An example is illustrated in Exhibit 10-18. In the absence of a parallel managed lane facility, the general purpose segment in the exhibit would be treated as one four-lane weaving segment with adjacent basic segments. However, because segmentation also needs to consider the managed lane segment types, and because adjacent lane groups need to be of equal length, the segmentation of the general purpose lane group is as follows: merge (Segment 1), basic (Segment 2), and diverge (Segment 3). The corresponding managed lane segments are categorized as ML diverge, ML basic, and ML basic, respectively.

Exhibit 10-18
Graphical Illustration of the Managed Lane Segmentation Method

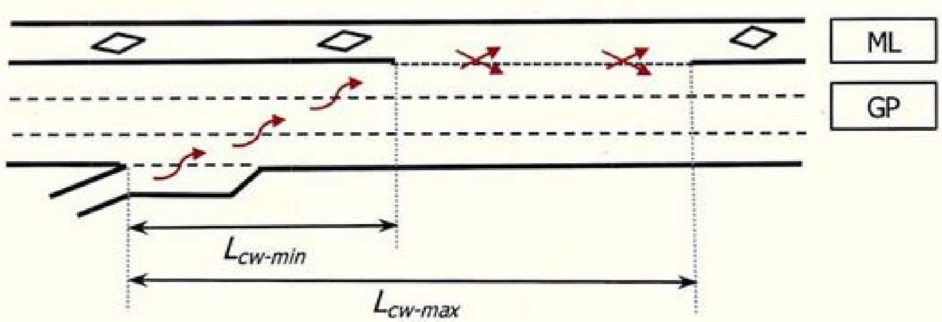


This example illustrates that the analyst may need to make compromises in the segmentation process when a general purpose lane with an adjacent managed lane is analyzed. In this case, evaluation of the general purpose lanes in isolation is also recommended to explore whether their performance changes significantly in moving from one (long) weaving segment to three separate segments. If substantial differences exist, the analyst should use capacity and speed adjustment factors (CAFs and SAFs) to calibrate the performance of these three segments and match the results to those of a general purpose–only analysis.

Cross-Weave Friction Effect

Where managed lanes have intermittent at-grade access from the general purpose lanes, a cross-weave movement may be created as vehicles entering the general purpose facility have to cross multiple lanes to reach the ML access segment. The ML access segment, in turn, is analyzed as a weaving segment to capture its friction. However, the cross-weave friction factor is applied to the general purpose segment(s) upstream of the actual access point. Exhibit 10-19 illustrates this cross-weave situation.

Exhibit 10-19
Cross-Weave Movement Associated with Managed Lane Access and Egress



Note: ML = managed lane, GP = general purpose.

Exhibit 10-19 illustrates a freeway facility consisting of a managed lane and three general purpose lanes. Where a general purpose merge is near an ML access segment, on-ramp vehicles destined for the managed lane must cross all of the general purpose freeway lanes in the distance $L_{cw\text{-}min}$. The cross-weave demand can cause a reduction in the capacity of the general purpose lanes, which must be considered. While not shown, the same effect exists when an off-ramp is near the ML access segment, with the distance $L_{cw\text{-}min}$ measured from the end of the access segment to the off-ramp junction point.

This effect is different from the weaving turbulence that occurs within the ML access segment, as vehicles entering and exiting from the managed lane cross paths within the distance $L_{cw\text{-}max} - L_{cw\text{-}min}$.

In estimating general purpose segment capacity, the cross-weave adjustment should be taken into account to quantify the reduction in general purpose segment capacity as a result of significant managed lane cross-weave flows. The adjustment should be applied where there is intermittent access to the managed lane over an access segment. A comprehensive methodology is provided in Chapter 13, Freeway Weaving Segments, to account for cross-weave capacity reduction on the general purpose lanes.

Adjacent Friction Effect

The adjacent friction effect applies when the general purpose lane group operates at densities above a specified threshold. Research has shown that managed lane operations are affected by these high general purpose lane densities in cases where no physical separation exists between the two facilities. For physically separated managed lanes, no adjacent friction effect applies.

For managed lanes without physical separation, a friction-constrained speed prediction model is used to estimate managed lane speeds. When the general purpose lanes operate below the specified density threshold, the non-friction-based speed prediction model is used. This factor is applied to both Continuous Access and Buffer 1 basic managed lane segments. Additional discussion of this effect is provided in Chapter 12.

Computational Steps

The computational steps for a managed lane analysis are largely consistent with the analysis of general purpose lanes. Several additional steps apply, which were highlighted in Exhibit 10-8 and described in Section 3. Specifically, the four unique computational steps for the managed lane extension are as follows:

- Step A-9: Managed Lane Cross-Weave Adjustment,
- Step A-13: Apply Managed Lane Adjacent Friction Factor,
- Step A-14: Compute Lane Group Performance, and
- Step A-17: Estimate LOS and Report Performance Measures for Lane Groups and Facility.

Of these four steps, only the first has to be applied manually by the analyst. The other three are performed automatically by a computational engine.

ACTIVE TRAFFIC AND DEMAND MANAGEMENT

The evaluation of ATDM strategies is described in detail in Chapter 11, Freeway Reliability Analysis. In that chapter, the effects of ATDM strategies such as ramp metering and hard-shoulder running are described in the context of a whole-year reliability analysis that covers a range of conditions. Chapter 11's methodology is the recommended way for evaluating ATDM strategies as part of a whole-year analysis.

However, an analyst may also be interested in evaluating the effects of a specific ATDM strategy on a single "representative" day (study period). Similar to the 1-day work zone analysis extension discussed above, a single study period ATDM analysis may provide insights into the relative effects of various strategies, such as when ATDM investments are compared with geometric improvements on the facility.

Chapter 37, ATDM: Supplemental, provides an overview of different ATDM strategies and guidance on their expected effects on facility performance. The analyst may use the available calibration metrics for freeway facilities, including capacity, speed, and demand adjustment factors (CAFs, SAFs, and DAFs) to estimate the effects of those strategies on the facility. The following list provides examples of other types of strategy assessments that can be performed by using this chapter's methodology:

1. A growth factor effect can be added to evaluate traffic performance when traffic demands are higher or lower than the demand calculated from the traffic counts. This parameter would be used to undertake a sensitivity analysis of the effect of demand on freeway performance and to evaluate future scenarios. In these cases, all cell demand estimates are multiplied by the growth factor parameter.
2. The effect of a predetermined ramp-metering plan can be evaluated by modifying the ramp roadway capacities. The capacity of each entrance ramp in each time interval is changed to the desired metering rate. This feature permits evaluation of a predetermined ramp-metering plan and experimentation to obtain an improved ramp-metering plan.
3. Freeway design improvements can be evaluated with this methodology by modifying the design features of any portion of the freeway facility. For example, the effect of adding an auxiliary lane at a critical location or of adding merging or diverging lanes can be assessed.
4. Reduced-capacity situations can be investigated. The capacity in any cell or cells of the time–space domain can be reduced to represent situations such as construction and maintenance activities, adverse weather, and traffic accidents and vehicle breakdowns.
5. User-demand responses, such as spatial, temporal, modal, and total demand responses caused by a traffic management strategy, are not automatically incorporated into the methodology. On viewing the new freeway traffic performance results, the user can modify the demand input manually to evaluate the effect of anticipated demand responses.

5. APPLICATIONS

Specific computational steps for the freeway facility methodology were conceptually discussed and presented in this chapter's methodology section. Computational details are provided in Chapter 25, Freeway Facilities: Supplemental.

This chapter's methodology is sufficiently complex to require the use of software for its application. Even for fully undersaturated analyses, the number and complexity of computations make manual analysis of a case difficult and extremely time-consuming. Oversaturated analyses are considerably more complex, and manual solutions are impractical. A computational engine and accompanying user's guide are available in Volume 4 for research purposes but should not be used for commercial applications.

EXAMPLE PROBLEMS

Chapter 25, Freeway Facilities: Supplemental, provides six example problems that illustrate the steps in applying the core methodology to a freeway facility under a variety of conditions. Other examples illustrate the work zone and managed lane extensions, as well as the freeway facility planning methodology. Exhibit 10-20 shows the list of example problems.

Exhibit 10-20
List of Example Problems

Example Problem	Description	Application
1	Evaluation of an undersaturated facility	Operational analysis
2	Evaluation of an oversaturated facility	Operational analysis
3	Capacity improvements to an oversaturated facility	Operational analysis
4	Evaluation of an undersaturated facility with work zone	Operational analysis
5	Evaluation of an oversaturated facility with managed lanes	Operational analysis
6	Planning-level evaluation of a freeway facility	Planning analysis

RELATED CONTENT IN THE HCMAG

The *Highway Capacity Manual Applications Guide* (HCMAG), accessible through the online HCM Volume 4, provides guidance on applying the HCM for freeway facility analyses. Case Study 4 goes through the process of identifying the goals, objectives, and analysis tools for investigating LOS on New York State Route 7, a 3-mi route north of Albany. The case study applies the analysis tools to assess the performance of the route, to identify areas that are deficient, and to investigate alternatives for correcting the deficiencies.

This case study includes the following problems related to basic freeway segments:

1. Problem 4: Analysis of a freeway facility
    a. Subproblem 4a: Separation of Alternate Route 7 for HCM analysis
    b. Subproblem 4b: Study of off-peak periods
    c. Subproblem 4c: What is the operational performance of Alternate Route 7 during the peak period?

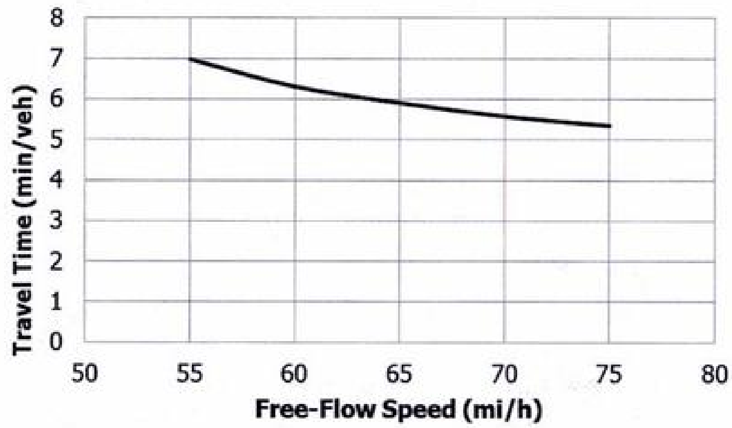
Although the HCMAG was based on the HCM2000's procedures and chapter organization, the general thought process described in its case studies continues to be applicable to current HCM methods.

EXAMPLE RESULTS

This section presents the results of applying this chapter's methodology in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs, as well as to help evaluate whether their results are reasonable. The exhibits in this section are not intended to substitute for an actual analysis and are deliberately provided in a format large enough to depict general trends in the results but not large enough to pull out specific results.

Total travel time on a freeway facility is sensitive to a number of input variables, including the prevailing FFS, demand levels, segment capacity, percentage drop in queue discharge rate, and demand-to-capacity ratio. Exhibit 10-21 illustrates the resulting facility-level travel time for values of FFS ranging from 55 to 75 mi/h for an example 6-mi-long facility (Example Problem 1 in Chapter 25). As apparent from the exhibit, an increase in the freeway facility FFS yields a reduction in the travel time. This result is due to the close association between capacity and FFS, with higher FFS values generating higher capacities and consequently lower travel times.

Exhibit 10-21
Facility Travel Time Sensitivity to Free-Flow Speed



Demand levels and the capacities of different segments along a freeway facility also influence total travel time. An overall increase in the demand level is expected to increase facility travel time, while an overall increase in segment capacity is expected to reduce travel time. Furthermore, an overall increase in the demand-to-capacity ratio is expected to increase travel time.

Exhibit 10-22 illustrates facility travel time sensitivity to changes in the demand-to-capacity ratio of the critical segment of a freeway facility. Specifically, the demand-to-capacity ratio is increased from 0.65 to as high as 1.4 on the last segment of Example Problem 1 in Chapter 25.

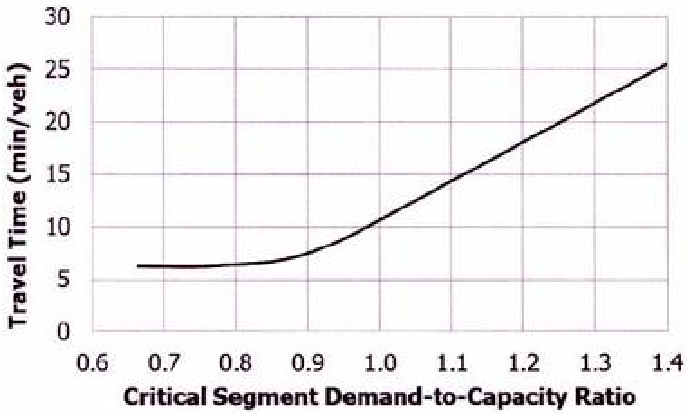


Exhibit 10-22
Facility Travel Time Sensitivity to *d/c* Ratio on Critical Segment

As apparent from the exhibit, increasing the demand-to-capacity ratio results in a gradual increase in facility travel time in undersaturated conditions; however, when demand exceeds the capacity ($d/c > 1.0$), travel time increases at a higher rate with an increase in the demand-to-capacity ratio.

A change in the percentage drop in capacity, modeling the effect of postbreakdown queue discharge rate, is also expected to influence travel time on a freeway facility. A larger drop in the queue discharge rate yields a longer travel time across the facility, as shown in Exhibit 10-23. Example Problem 2 in Chapter 25 was used to generate this exhibit. This result occurs because higher capacity drops mean that when oversaturation occurs, queues will build up faster and recover more slowly as the queue discharge rate is lowered.

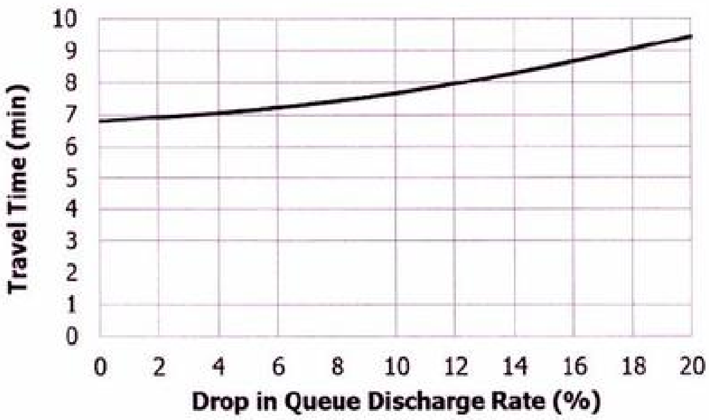


Exhibit 10-23
Facility Travel Time Sensitivity to Percentage Drop in Queue Discharge Rate

PLANNING, PRELIMINARY ENGINEERING, AND DESIGN ANALYSIS

The operational methodology for freeway facilities cannot be readily adapted to planning, preliminary engineering, and design applications because of the amount of data required and the method's computational complexity. However, a separate planning methodology is available for evaluating a freeway facility in a planning context. The methodology is based on national research (*11*) and is calibrated to approximate the results of an operational analysis, but with reduced input needs and computational burden. The method is introduced below and described in more detail in Chapter 25: Freeway Facilities: Supplemental.

Service Volume Tables

The service volume tables provided in Chapter 12 for basic freeway segments can be used to obtain a quick planning-level estimate of the service volumes that can be supported on a freeway. These tables may be applied for general evaluations of a number of freeway facilities in a specified region. They should not be used for directly evaluating a specific freeway facility or for developing detailed facility improvement plans. A full operational analysis would normally be applied to any freeway facility identified as potentially needing improvement, with the planning methodology providing an alternative with reduced data input needs and computational time.

Segment-Based Planning Applications

The segment procedures described in Chapters 12, 13, and 14 can also be used in preliminary engineering and design applications of the methodology. Various geometric scenarios can be evaluated and compared by using a travel demand matrix and applying the facility methodology to each scenario's segment results.

Freeway Facility Planning Method

For planning applications, a simplified planning-level methodology may be desirable (*11*). The approach is based on and compatible with this chapter's operational methodology, but the planning method is specifically constructed to minimize input data requirements. The planning method covers both undersaturated and oversaturated flow conditions and produces estimates of travel time, speed, density, and level of service. The method is based on the use of sections rather than segments, with a section being defined as the distance between two ramp gore points. Section breaks also occur when lanes are added or dropped. The underlying methodology relies on developing a relationship between the delay rate per unit distance on a basic freeway segment and the segment's demand-to-capacity ratio.

For weaving sections, the method applies capacity adjustment factors on the basis of the volume ratio and segment length. With these factors, a weaving section's demand-to-capacity ratio is adjusted and the segment is then treated similarly to a basic freeway segment.

For ramp sections with merge or diverge segments, or both, the methodology estimates the segment capacity on the basis of the demand level, free-flow speed, and space mean speed. Capacity adjustment factors are then calculated for these sections, and their demand-to-capacity ratios are adjusted accordingly.

The methodology first estimates demand-to-capacity ratios for each section. In oversaturated conditions, the number of vehicles queued on a section in one analysis period is added to its demand in the next analysis period, and demand-to-capacity ratios are adjusted accordingly.

The freeway facility planning method is discussed in detail in Chapter 25, Freeway Facilities: Supplemental.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for applying alternative tools to the analysis of freeway facilities. Additional information on this topic may be found in Chapter 25, Freeway Facilities: Supplemental.

Strengths of the HCM Procedure Compared with Alternative Tools

This chapter's procedures were based on extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent a consensus of experts. Specific strengths of the HCM freeway facilities procedures include the following:

- They provide more detailed algorithms for considering geometric elements of the facility (such as lane and shoulder width).
- They provide capacity estimates for each segment of the facility, which simulation tools do not provide directly (and in some cases may require as an input).
- The capacity can be explicitly adjusted to account for weather conditions, lighting conditions, work zone setup and activity, and incidents.
- The calculation of key performance measures, such as speed and density, is transparent. Simulation tools often use statistics accumulated over the simulation period to derive various link- or time-period-specific results, and the derivation of these results may not be obvious. Thus, the user of a simulation tool must know exactly which measure is being reported (e.g., space mean speed versus time mean speed). Furthermore, simulation tools may apply these measures in ways different from the HCM to arrive at other measures.

Limitations of the HCM Procedures That Might Be Addressed with Alternative Tools

Freeway facilities can be analyzed with a variety of stochastic and deterministic simulation tools. These tools can be useful in analyzing the extent of congestion when there are failures within the simulated facility range and when interaction with other freeway segments and other facilities is present.

Exhibit 10-24 provides a list of the limitations stated earlier in this chapter, along with their potential for improved treatment by alternative tools.

Exhibit 10-24
Limitations of the HCM Freeway Facilities Analysis Procedure

Limitation	Potential for Improved Treatment with Alternative Tools
Changes in travel time caused by vehicles using alternate routes	Modeled explicitly by dynamic traffic assignment tools
Multiple overlapping bottlenecks	Modeled explicitly by simulation tools
User-demand responses (spatial, temporal, modal)	Modeled explicitly by dynamic traffic assignment tools
Systemwide oversaturated flow conditions	Modeled explicitly by simulation tools
First/last time interval or first/last segment demand-to-capacity ratio > 1.0	Modeled explicitly by simulation tools, except that a simulation analysis may also be inaccurate if it does not fully account for a downstream bottleneck that causes congestion in the last segment during the last time period
Interaction between managed lanes and mixed-flow lanes	Modeled explicitly by some simulation tools

Additional Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the following performance measures for individual segments along a freeway facility and for the entire facility, given each segment's traffic demand and characteristics:

- Travel time,
- Free-flow travel time,
- Traffic delay,
- Vehicle miles of travel,
- Person miles of travel,
- Speed, and
- Density (segment only).

Alternative tools can offer additional performance measures, such as queue lengths, fuel consumption, vehicle emissions, operating costs, and vehicle acceleration and deceleration rates. As with other procedural chapters in the HCM, simulation outputs—especially graphics-based outputs—may provide details on point problems that might go unnoticed with a macroscopic analysis.

Development of HCM-Compatible Performance Measures Using Alternative Tools

LOS for all types of freeway segments is estimated by the density of traffic (pc/mi/ln) on each segment. The guidance provided in Chapter 11, Basic Freeway Segments, for developing compatible density estimates applies to freeway facilities as well.

With the exception of free-flow travel time, the additional performance measures listed above that are produced by the procedures in this chapter are also produced by typical simulation tools. For the most part, the definitions are compatible, and, subject to the precautions and calibration requirements that follow, the performance measures from alternative tools may be considered equivalent to those produced by the procedures in this chapter.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

To determine when simulation of a freeway facility may be more appropriate than an HCM analysis, the fundamental differences between the two approaches must be understood. The HCM and simulation analysis approaches are reviewed in the following subsections.

HCM Approach

The HCM analysis procedure uses one of two approaches—one for undersaturated conditions and one for oversaturated conditions. For the former—that is, v_d/c is less than 1.0 for all segments and analysis time periods—the approach is generally disaggregate. In other words, the facility is subdivided into segments corresponding to basic freeway, weaving, and merge or diverge segments, and the LOS results are reported for individual segments on the basis of the analysis procedures of Chapters 12–14. LOS results are aggregated for the facility as a whole in each analysis period.

For oversaturated conditions, the facility is analyzed in a different manner. First, the facility is considered in its entirety rather than at the individual segment level. Second, the analysis time interval, typically 15 min, is subdivided into time steps of 15 s. This approach is necessary so that flows can be reduced to capacity levels at bottleneck locations and queues can be tracked in space and time. The average density of an oversaturated segment is calculated by dividing the average number of vehicles in the segment across these time steps by the segment length. The average segment speed is calculated by dividing the average segment flow rate by the average segment density. Facilitywide performance measures are calculated by aggregating segment performance measures across space and time, as outlined in Chapter 25. A LOS for the facility is assigned on the basis of density for each time interval.

When the oversaturation analysis procedure is applied, if any segment is undersaturated for an entire time interval, its performance measures are calculated according to the appropriate procedure in Chapters 12–14.

Simulation Approach

Simulation tools model the facility in its entirety and from that perspective have some similarity to the oversaturated analysis approach of the HCM. Microscopic simulation tools operate similarly under saturated and undersaturated conditions. They track each vehicle through time and space and generally handle the accumulation and queuing of vehicles in saturated conditions in a realistic manner. Macroscopic simulation tools vary in their treatment of saturated conditions. Some tools do not handle oversaturated conditions at all; others may queue vehicles in the vertical rather than the horizontal dimension. These tools may still provide reasonably accurate results under slightly oversaturated conditions, but the results will clearly be invalid for heavily congested conditions.

The treatment of oversaturated conditions is a fundamental issue that must be understood in considering whether to apply simulation in lieu of the HCM for analysis of congested conditions. A review of simulation modeling approaches is beyond the scope of this document. More detailed information on the topic may be found in the Technical Reference Library in Volume 4.

Adjustment of Simulation Parameters to the HCM Results

Some calibration is generally required before an alternative tool can be used effectively to supplement or replace the HCM procedure. The following subsections discuss key variables that should be checked for consistency with the HCM procedure values.

Capacity

In the HCM, prebreakdown capacity is a function of the specified or computed free-flow speed (which can be adjusted by lane width, shoulder width, and ramp density) and of capacity adjustment factors that account for local conditions, driver population effects, weather, incidents, and work zones. In a simulation tool, capacity is typically a function of the specified minimum vehicle entry headway (into the facility) and car-following parameters (if the discussion pertains to microscopic simulation).

In macroscopic simulation tools, capacity is generally an input. For this situation, matching the simulation capacity to the HCM capacity is straightforward. However, microscopic simulation tools do not have an explicit capacity input. Most microscopic tools provide an *input that affects* the minimum separation for the generation of vehicles into the system. Specifying a value of 1.5 s for this input will result in a maximum vehicle entry rate of 2,400 (3,600/1.5) veh/h/ln. Once vehicles enter the system, vehicle headways are governed by the car-following and gap acceptance models. In view of other factors and model constraints, the maximum throughput on any one segment may not reach this value. Consequently, some experimenting is usually necessary to find the right minimum entry separation value to achieve a capacity value comparable with that in the HCM. Again, the analyst needs to be mindful of the units being used for capacity in making comparisons.

The other issue to be aware of is that, while geometric factors such as lane and shoulder width affect the free-flow speed (which in turn affects capacity) in the HCM procedure, some simulation tools do not account for these effects, or they may account for other factors, such as horizontal curvature, that the HCM procedure does not consider.

Lane Distribution

In the HCM procedure, there is an implicit assumption that, for any given vehicle demand, the vehicles are evenly distributed across all lanes of a basic freeway segment. For merge and diverge segments, the HCM procedure includes calculations to determine how vehicles are distributed across lanes as a result of merging or diverging movements. For weaving segments, there is not an explicit determination of flow rates in particular lanes, but consideration of weaving and nonweaving flows and the number of lanes available for each is an essential element of the analysis procedure.

In simulation tools, the distribution of vehicles across lanes is typically specified only for the entry point of the network. Once vehicles have entered the network, they are distributed across lanes according to car-following and lane-changing logic. This input value should reflect field data if they are available. If field data indicate an imbalance of flows across lanes, a difference between the HCM and simulation results may ensue. If field data are not available, specifying an even distribution of traffic across all lanes is probably reasonable for networks that begin with a long basic segment. If there is a ramp junction within a short distance downstream of the entry point of the network, setting the lane distribution values to be consistent with those from Chapter 14 of the HCM will likely yield more consistent results.

Traffic Stream Composition

The HCM deals with the presence of non–passenger car vehicles in the traffic stream by applying passenger car equivalent values. These values are based on the percentage of single-unit trucks, buses, and tractor-trailers in the traffic stream, as well as the type of terrain (grade profile and its length). The values also depend on the relative heavy vehicle fleet mix between single-unit trucks (including buses and recreational vehicles) and tractor-trailer trucks. Thus, the traffic stream is converted into some equivalent number of passenger cars only, and the analysis results are based on flow rates in these units.

In the case of stochastic-based simulators, the generated vehicle type percentages may only approximate the specified percentages.

Simulation tools deal with the traffic stream composition just as it is specified; that is, the specific percentages of each vehicle type are generated and moved through the system according to their specific vehicle attributes (e.g., acceleration and deceleration capabilities). Thus, simulation, particularly microscopic simulation, results likely better reflect the effects of non–passenger car vehicles on the traffic stream. Although in some instances the HCM's passenger car equivalent values were developed from simulation data, simplifying assumptions made to implement them in an analytical procedure result in some loss of fidelity in the treatment of different vehicle types.

In addition, HCM procedures do not explicitly account for differences in driver types. Microscopic simulation tools explicitly provide for a range of driver types and allow a number of factors related to driver type to be modified (e.g., FFS, gap acceptance threshold). However, the empirical data supporting some HCM procedures include the effects of the various driver types present in traffic streams.

Free-Flow Speed

In the HCM, FFS is either measured in the field or estimated with calibrated predictive algorithms. In simulation, FFS is almost always an input value. Where field measurements are not available, simulation users may wish to use the HCM predictive algorithms to estimate FFS.

Step-by-Step Recommendations for Applying Alternative Tools

General guidance for applying alternative tools is provided in Chapter 6, HCM and Alternative Analysis Tools. The chapters that cover specific types of freeway segments offer more detailed step-by-step guidance specific to those segments. All the segment-specific guidance applies to freeway facilities, which are configured as combinations of different segments.

The first step is to determine whether the facility can be analyzed satisfactorily by the procedures described in this chapter. If the facility contains geometric or operational elements beyond the scope of these procedures, an alternative tool should be selected. The steps involved in the application will depend on the reason(s) for choosing an alternative tool. In some cases, the step-by-step segment guidance will cover the situation adequately. In more complex cases (e.g., those involving integrated analysis of a freeway corridor), more comprehensive guidance from one or more documents in the Technical Reference Library in Volume 4 may be needed.

Sample Calculations Illustrating Alternative Tool Applications

The limitations of this chapter's procedures are mainly related to the lack of a comprehensive treatment of the interaction between segments and facilities and between facilities, for example a freeway and parallel surface street arterial forming a corridor. Many of these limitations can be addressed by simulation tools, which generally take a more integrated approach to the analysis of complex networks of freeways, ramps, and surface street facilities. Supplemental examples illustrating interactions between segments are presented in Chapter 26, Freeway and Highway Segments: Supplemental, and Chapter 34, Interchange Ramp Terminals: Supplemental. A comprehensive example of the application of simulation tools to a major freeway reconstruction project is presented as Case Study 6 in the HCM Applications Guide located in Volume 4.

6. REFERENCES

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

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CHAPTER 11
FREEWAY RELIABILITY ANALYSIS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

This chapter provides a methodology for evaluating a freeway's travel time reliability over a multiday or multimonth *reliability reporting period* (RRP). The methodology estimates the impacts of recurring and nonrecurring congestion (i.e., demand variations and fluctuations, incidents, weather, work zones, and special events) on the travel time distribution over the course of the RRP. The methodology can be extended to estimate the impacts of active traffic and demand management (ATDM) strategies on the travel time distribution.

The methodology relies on the freeway facilities core methodology presented in Chapter 10, which in turn applies the freeway segment methodologies in Chapters 12, 13, and 14. The freeway facilities core methodology focuses the analysis on a single day or less, while the segment methodologies are limited to the analysis of one 15-min period. In contrast, this chapter's methodology is capable of applying the core method repeatedly across multiple days, weeks, and months, up to a 1-year RRP. A 1-year RRP is the most common application, although shorter periods are possible for specific applications (e.g., reliability of summer tourist traffic, a focus on the construction season). RRPs longer than 1 year are uncommon, since most typical variations in travel time (day of week, month of year, weather, and incidents) are encapsulated in a single year.

The methodology is integrated with the FREEVAL-2015E computational engine, which implements the complex computations involved. This engine was developed to test the methodology; other software implementations are available. This chapter discusses the basic principles of the methodology and its application. Chapter 25, Freeway Facilities: Supplemental, provides a detailed description of all the algorithms that define the methodology.

CHAPTER ORGANIZATION

Section 2 of this chapter presents the basic concepts of freeway reliability analysis, including performance measures derived from the travel time distribution. The section also provides an introduction to scenario generation concepts and evaluation of ATDM strategies in the context of this chapter.

Section 3 presents the base methodology for evaluating freeway reliability. The method generates a series of performance measures that can be derived from the travel time distribution, including various percentile travel time indices and on-time performance ratings.

Section 4 extends the core method presented in Section 3 to the evaluation of ATDM strategies in a travel time reliability context.

Section 5 presents guidance on using the results of a freeway facility analysis, provides example results from the methods, discusses planning-level reliability analysis, and provides guidance on the use of alternative tools.

RELATED HCM CONTENT

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

- Chapter 3, Modal Characteristics, where the motorized vehicle methodology's Variations in Demand subsection describes typical travel demand patterns for freeway and multilane highway segments;
- Chapter 4, Traffic Operations and Capacity Concepts, which provides background for the refinements specific to freeway and multilane highway segments that are presented in this chapter's Section 2;
- Chapter 10, Freeway Facilities Core Methodology, which forms the basis for this chapter's computations in a single-day application;
- Chapters 12, 13, and 14, which present the segment methodologies for basic freeway segments, freeway weaving segments, and freeway merge and diverge segments, respectively;
- Chapter 25, Freeway Facilities: Supplemental, which provides the computational details of this chapter's methodology, including a detailed description of the scenario generation procedure;
- Chapter 37, ATDM: Supplemental, which provides additional details and concepts related to ATDM strategy types and their expected operational impacts; and
- Section H, Freeway Analyses, in the *Planning and Preliminary Engineering Applications Guide to the HCM,* found in Volume 4, which describes how to incorporate this chapter's methods and performance measures into a planning effort.

2. CONCEPTS

OVERVIEW

Freeway travel time reliability reflects the distribution of the travel times for trips traversing an entire freeway facility over an extended period of time, typically 1 year, during any portion of the day. A 1-year RRP is typical, since it covers most variation in travel times arising from the factors below. Shorter RRPs are possible for special circumstances, such as a focus on summer tourist travel or the work zone construction season.

The travel time distribution is created by the interaction of several factors that influence facility travel times:

Travel time reliability is influenced by demand variations, weather, incidents, work zones, and special events.

- *Recurring variations in demand* by hour of day, day of week, and month of year; within certain limits, these variations are more or less predictable;
- *Severe weather* (e.g., heavy rain, snow, poor visibility) that reduces speeds and capacity and may influence demand; this is a nonrecurring event;
- *Incidents* (e.g., crashes, disabled vehicles, debris) that reduce capacity; these are nonrecurring events;
- *Work zones* that reduce capacity and—for longer-duration work activities—may influence demand; these are nonrecurring events; and
- *Special events* that produce temporary intense traffic demands, which may be managed in part by changes in the facility's geometry or traffic control; special events can be scheduled or recurring (e.g., a state fair) or nonrecurring (e.g., concerts).

As explained in the Travel Time Reliability section of Chapter 4, Traffic Operations and Capacity Concepts, the underlying distribution of travel times expresses the *variability* in travel times that occur on a facility or a trip over the course of time, as expressed by 50th, 80th, and 95th percentile travel times and other distribution metrics. The travel time observations in the distribution are the *average* facilitywide travel times over a 15-min period, not individual vehicle travel times.

ATDM for freeways consists of the dynamic and continuous monitoring and control of traffic operations to improve facility performance. Examples of freeway ATDM measures are managed lanes, dynamic ramp metering, incident management, changeable message signs, hard shoulder running, and speed harmonization (variable speed limits). ATDM strategies are discussed in detail in Chapter 37, ATDM: Supplemental.

ATDM consists of the dynamic and continuous monitoring and control of traffic operations.

ATDM measures can influence both the nature of demand on the freeway facility and the ability of the facility to deliver the capacity and quality of service tailored to serve the demand. Combining reliability and ATDM in this chapter is natural, since the ATDM toolbox serves to mitigate nonrecurring congestion in a near-real-time, dynamic response mode.

In a highway capacity analysis context, the effects of both (*a*) factors affecting travel time reliability (e.g., weather, incidents) and (*b*) ATDM strategies are modeled as variations in (or adjustments to) one or more parameters used in a freeway facilities analysis. These parameters are adjusted during specific time periods on specific affected freeway segments. These parameters include

- Number of mainline lanes open to traffic;
- Available capacity per freeway lane that is open to traffic;
- Facility free-flow speed;
- On-ramp capacity or throughput;
- Demand flow rates at origin points, destination points, or both;
- Incident frequencies; and
- Incident clearance times.

FREEWAY TRAVEL TIME AND RELIABILITY

Objectives for Reliability Analysis

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

- *Tracking* the reliability of a set of freeway facilities in a jurisdiction or region over an extended period to prioritize operational or physical strategies intended to improve reliability;
- *Diagnosing* the primary causes of the reliability problems on a given facility so that an improvement program can be developed and specific strategies applied to enhance reliability; and
- *Predicting* the effects of a particular treatment or improvement strategy on a facility, including testing the effectiveness and benefit–cost of ATDM strategies.

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

More broadly, travel time reliability analysis can be used to improve the operation, planning, prioritization, and programming of transportation improvement projects in the following applications: long-range transportation plans, transportation improvement programs, corridor plans, major investment studies, congestion management, operations planning, and demand forecasting.

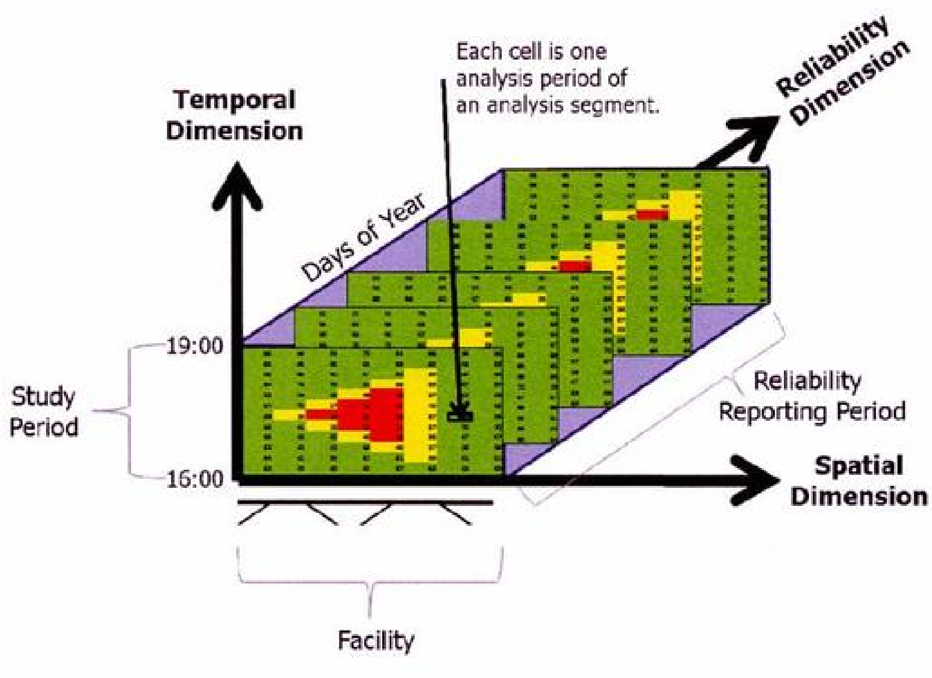
Reliability analyses can often also be performed by using field data gathered through the use of sensors and stored in long-term speed and travel time archives increasingly available to many transportation agencies. The HCM reliability method can be used to supplement these field sources and is particularly valuable in evaluating and testing strategies intended to improve reliability, as discussed in the bullets above. Field data can also be used to validate the HCM method, but the method described in this chapter is uniquely suited for evaluating trade-offs and the benefit–cost relationship of different strategies intended to make a facility more reliable.

Reliability Methodology Definitions

Conceptually, travel time reliability can be viewed as an extension of the freeway facilities core methodology presented in Chapter 10. The extension occurs in the time dimension, by transitioning from a "typical day" or "single study period" analysis to a reliability dimension, which is an extended-period analysis covering several days, weeks, months, or a full year. This new dimension gives rise to the following set of definitions, many of which are illustrated in Exhibit 11-1:

- *Travel time.* The time required for a vehicle to travel the full length of the freeway facility from mainline entry point to mainline exit point without leaving the facility or stopping for reasons unrelated to traffic conditions.
- *Free-flow travel time.* The facility's length divided by its free-flow speed.
- *Travel time index (TTI).* The ratio of the actual travel time to the free-flow travel time. By definition, TTI is always greater than or equal to 1.0. The TTI's distribution is identical to that of travel time, except that its values are indexed to the free-flow travel time.
- *Percentile travel time index (TTI_{pp}).* Represents the *pp* percentile TTI in the travel time distribution. For example, TTI_{85} means that this observation is exceeded only 15% of the time in the travel time distribution. Common TTI percentiles are TTI_{50} (the median TTI) and TTI_{95} (the 95th percentile TTI). When *pp* is omitted, the value often represents the *mean* TTI for the distribution, which in this chapter is referred to as TTI_{mean}.
- *Analysis segment.* An HCM freeway segment (e.g., basic, merge, diverge, weaving) as described in Chapters 12 through 14. Each column in Exhibit 11-1 represents an analysis segment.

Exhibit 11-1
Schematic Representation of Freeway Reliability Analysis Time–Space Domain



Source: Zegeer et al. (1).

A scenario is a unique combination of traffic demand, capacity, geometry, and free-flow speed conditions for a given study period.

- *Analysis period.* The time interval evaluated by a single application of an HCM methodology (15 min for the freeway facilities core methodology). In Exhibit 11-1, there are 12 such analysis periods for the facility, represented by the rows in the rectangles. Each cell in a rectangle represents a single analysis period for a single analysis segment.
- *Study period.* The time interval within a day for which facility performance is evaluated. It consists of one or more consecutive analysis periods, represented by the rows in the rectangles in Exhibit 11-1. In this example, the study period is 3 h long, from 4 to 7 p.m. (i.e., 16:00 to 19:00 hours).
- *Scenario.* A single instance of a study period for the facility, with a unique combination of traffic demands, capacities, geometries, and free-flow speeds represented in its analysis periods. Each rectangle in Exhibit 11-1 represents a unique scenario, or in other words 1 day of the year.
- *Base scenario (seed file).* A set of parameters representing the facility's calibrated operating conditions during one study period. All other scenarios are developed by adjusting the base scenario's inputs to reflect the effects of varying demand, weather, incidents, work zones, or a combination occurring in other study periods. When the methodology is executed by using a computational engine, the base scenario's parameters become inputs to the seed file used by the engine.
- *Reliability reporting period.* The specific set of days over which travel time reliability is computed; for example, all nonholiday weekdays in a year. The RRP represents the third dimension that extends the freeway facilities core methodology and is illustrated in Exhibit 11-1 by the series of rectangles (scenarios).
- *Travel time distribution.* The distribution of average facility travel times by analysis period across the RRP. Each 15-min analysis period within each scenario contributes one data point to the travel time distribution. It is *not* the distribution of individual vehicle travel times (or TTIs).
- *Probability density function (PDF) and cumulative distribution function (CDF).* The PDF gives the number or percent of all observations within a specified travel time (or TTI) bin. The CDF gives the number or percent of all observations *at or below* a specified travel time bin. Exhibit 11-2 illustrates the two types of distributions, with the PDF shown by the solid line and the CDF by the dashed line. The facility travel times shown on the x-axis are the midpoints of the various travel bins.

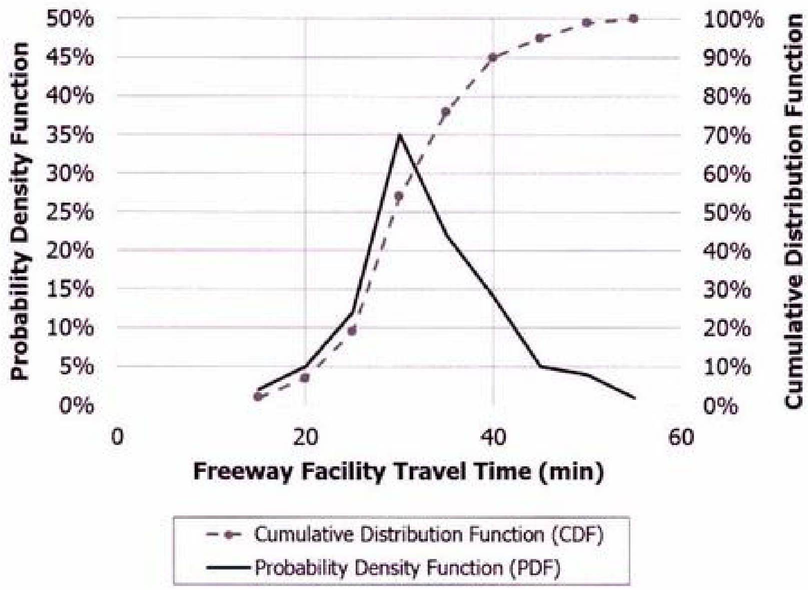


Exhibit 11-2
Illustrative Probability Density and Cumulative Distribution Functions of Travel Time on a Freeway Facility

Travel Time Distribution and Reliability Performance Measures

Exhibit 4-5 in Chapter 4, Traffic Operations and Capacity Concepts, illustrates how various reliability performance measures can be derived from the CDF of a travel time distribution. When travel times are measured or predicted over a long period (e.g., a year), a distribution of travel time emerges. The following are useful measures for describing (*a*) travel time or TTI variability or (*b*) the success or failure of individual trips in meeting a target travel time or speed:

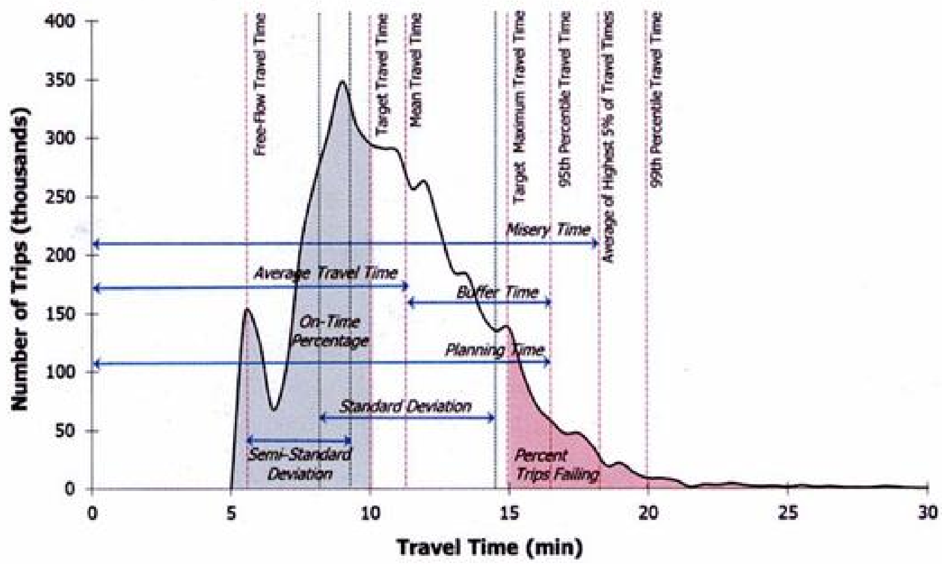
- TTI_{95} (unitless). The 95th percentile TTI is also referred to as the planning time index (PTI) and is a useful measure for estimating the added time travelers must budget to ensure an on-time arrival with "failure" limited to one trip per month. In Exhibit 11-2, the 95th percentile travel time is 45 min, compared with a free-flow travel time of 15 min; thus, $TTI_{95} = 3.0$. The planning time is the difference between the 95th percentile and free-flow travel times, or 30 min.
- TTI_{80} (unitless). Research indicates that this measure is more sensitive to operational changes than the TTI_{95} (2), which makes it useful for strategy comparison and prioritization purposes. In Exhibit 11-2, the 80th percentile travel time is approximately 36 min; thus the 80th percentile TTI is 36/15 = 2.4.
- TTI_{50} and TTI_{mean} (unitless). These measures describe the median and mean of the TTI distribution, respectively. Both can be useful measures, with the median being less influenced by outliers than the mean.
- *Failure* or *on-time measures* (percentage). The percentage of analysis time periods with space mean speeds above (on time) or below (failure) one or more target values (e.g., 35, 45, and 50 mi/h). These measures address how often trips succeed or fail in achieving a desired travel time or speed.
- *Reliability rating* (percentage). The percentage of vehicle miles traveled (VMT) on the freeway facility that experiences a TTI less than 1.33. This

threshold approximates the points beyond which travel times become much more variable (unreliable).

- *Semi–standard deviation* (unitless). A one-sided standard deviation, with the reference point being free-flow travel time (or TTI = 1) instead of the mean. It reflects the mean variability from free-flow conditions.
- *Standard deviation* (unitless). The standard statistical measure.
- *Misery index* (unitless). A measure comparing the average of the worst 5% of travel times with the free-flow travel time.

The travel time distribution and some of its key performance measures are illustrated in Exhibit 11-3.

Exhibit 11-3
Derivation of Time-Based Reliability Performance Measures from the Travel Time Distribution



Source: Zegeer et al. (1).

SCENARIO GENERATION

As the freeway facilities core methodology is expanded from a single study period (or representative condition) to capture variations in performance across the RRP, generation of scenarios describing how operations are affected by combinations of changes in demand, weather, incidents, and scheduled work zones becomes necessary. These factors are facility-specific. Weather depends on geographic location, incidents on congestion and incident management levels, work zone on infrastructure quality, facility demand on characteristics of the facility's travel patterns, and so on.

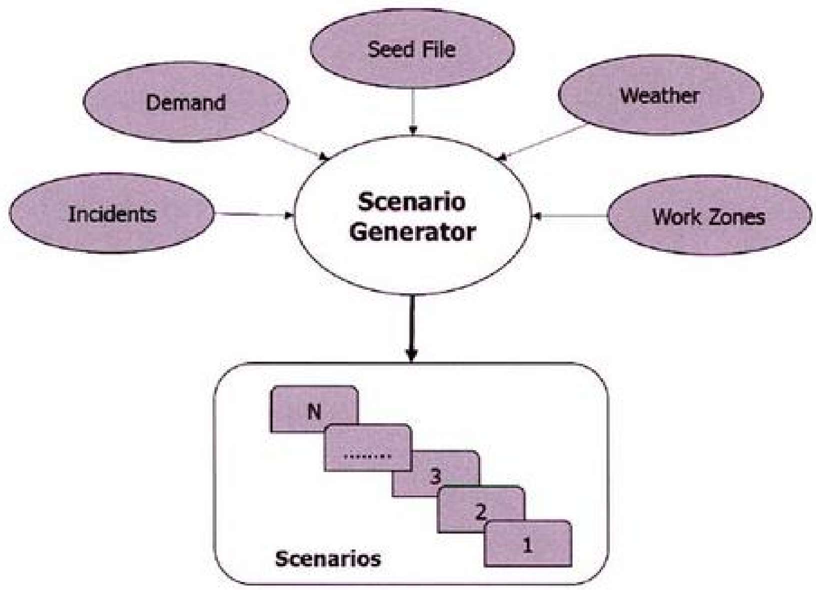
The process of enumerating the various combinations of these factors and calculating their probability of occurrence is termed *freeway scenario generation*. The scenario generation process is described conceptually in this section, and the step-by-step procedures for implementing freeway scenario generation are described in Chapter 25, Freeway Facilities: Supplemental.

The calendar creates an important connection between all the factors influencing travel time reliability. Weather is intuitively calendar-based (e.g., more snow falls in the winter than in the summer), as are traffic demand patterns to a great extent. Work zones, at least in areas with inclement winter weather, are typically scheduled to avoid extreme weather events. Furthermore, the number of incidents is likely to be directly correlated with traffic demand and thus indirectly tied to the calendar.

The calendar creates an important connection between all the reliability-affecting factors, such as weather, demand, incidents, and work zones.

The mechanism of implementing freeway scenario generation is actually simple. On the basis of the analyst's input of influential factors (e.g., how facility demand varies over time, how weather events vary on a monthly basis), the scenario generation process takes the input events and generates a combination of scenarios matching those inputs. All scenarios originate with the base scenario (seed file), whose inputs are manipulated via changes in free-flow speed, individual segment capacity, lane losses, and (possibly) demand changes to create a new unique combination of events, or scenario. A high-level schematic of the freeway scenario generation process is depicted in Exhibit 11-4.

Exhibit 11-4
Schematic of the Freeway Scenario Generation Process and Influential Factors



The *default* number of scenarios generated in this procedure, without considering weekends, is 240 (the parameter N in Exhibit 11-4). This value was obtained by creating four replications of each weekday–month demand combination (5 weekdays × 12 months × 4 replications). Having multiple replications of the weekday and month combination ensures inclusion of a sufficiently large sample of weather and incident events in the reliability analysis. Any stochastic scenario effects (e.g., weather or incidents) will vary across these four replications. Specific guidance for the number of replications as a function of the length of the RRP is given in Section 3, but the choice of four replications roughly corresponds to having each day of the week appear four times per month (e.g., approximately four Mondays in January). This process allows the procedure to produce a number of representative scenarios sufficient to accommodate the variability in all four factors affecting reliability. The analyst may increase the number of replications, since that parameter value in the procedure can be controlled by the user. This approach would be advisable if the RRP is short (e.g., a few weeks).

Scenario Generation Approach

The scenario generation process treats some factors affecting reliability deterministically and others stochastically.

The scenario generation procedure presented in this chapter is a *hybrid* approach, with some inputs being treated in a deterministic fashion and others being stochastic in nature. Traffic demand and scheduled work zones are treated in a deterministic manner. Direct calendar data are used to characterize demand variability (i.e., day of week, month of year), and user-defined work zone schedules are applied. On the other hand, weather and incidents are modeled in a stochastic fashion and are assigned randomly to scenarios. The assignment is based on predefined distributions of weather and incidents that the analyst specifies to describe the facility. When such data are not available or are incomplete, the method provides national default distributions to assist with the scenario generation process.

The objective of the scenario generation process is to maximize the match (or minimize the difference) between the generated scenarios and the input distributions of the factors affecting reliability, as entered by the user. This is accomplished by assigning the correct traffic demand levels, weather events, and incidents within the different scenarios. Eight distributions are entered into the freeway scenario generation procedure (*1*):

1. Temporal distribution of traffic demand levels,
2. Temporal distribution of weather event frequency,
3. Distribution of average weather event duration by weather event type,
4. Temporal distribution of incident event frequency,
5. Distribution of incident severity (i.e., shoulder, single, or multilane closures),
6. Distribution of incident duration by incident severity,
7. Distribution of incident event start time in a scenario, and
8. Spatial distribution of incident events across segments of the facility.

Only the first six distributions represent manual inputs by the user, and all have default values available. Items 7 and 8 in the list are estimated by the computational engine and do not require user input. Details on all distributions are provided in Chapter 25.

The hybrid freeway scenario generation approach optimizes the match between the generated events and the user inputs for demand, incidents, weather, and work zones.

Thus, the hybrid approach generates scenarios such that all eight specified distributions match actual conditions, with consideration for the need to round the number of events (incidents, weather, etc.) to integer values and to round their durations to the nearest 15-min analysis period.

Treatment of Factors Affecting Travel Time Reliability

This section provides a high-level description of how each of the four factors involved in the reliability analysis—demand, weather, incidents, and work zones—is treated in the scenario generation process.

Traffic Demand

The methodology accounts for demand variability by adjusting the traffic demands for the analysis periods included in the various scenarios. This is done through the use of a *demand multiplier*, which is the ratio of the daily (weekday–month combination) facility demand to the average daily traffic (or to any combination of day of week and month of year). A second adjustment is needed to factor the demand measured on the specific day–month combination *in the base scenario* to any other day–month combination in the year. Traffic demand variation for different hours of the day is already accounted for in the base scenario obtained from the Chapter 10 core facility analysis.

For example, if the *base scenario* demand data were gathered on a Monday in January that has a demand multiplier of 0.85 and a demand scenario is being tested on a Friday in June that has a demand multiplier of 1.10, the base scenario demands should be factored by a ratio of 1.10/0.85 = 1.29 to create the demand profile for that Friday-in-June scenario. If all days of the week are considered, there could be up to 84 demand combinations; for weekday-only analyses, there could be up to 60 demand combinations.

Weather Events

Weather events are generated on the basis of their probability of occurrence during a given month. The scenario generation process accounts for 10 categories of severe weather events that have been shown to reduce capacity by at least 4%, along with a *non–severe weather* category that encompasses all other weather conditions and that generates no capacity, demand, or speed adjustments. Default capacity and speed adjustment factors for weather events are provided in Section 5 of this chapter.

To capture the actual occurrences of various weather events, the analyst may use default weather data from any of 101 U.S. metropolitan areas, based on 2001–2010 weather records. These values are documented in the Volume 4 Technical Reference Library.

Alternatively, the analyst may supply a 12-month by 11-weather-event matrix (132 total values) of local probabilities of each weather event, along with the average duration for each event (10 values). As mentioned earlier, different weather events are assigned stochastically to the various scenarios in a manner that will match their monthly occurrence based on the site's meteorological data.

Traffic Incidents

Incidents are generated on the basis of their expected frequency of occurrence per study period (analysis hours in a day) in a given month on the facility. The analyst may opt to use default expected incident frequencies, may supply a facility-specific incident or crash rate, or may supply a 12-month table of facility-specific expected frequencies of any incident type. The incident frequency represents the average number of all incidents experienced on the facility during the study period and is allowed to vary in each month.

The method makes the following assumptions about a given incident:

- The incident start time is assigned stochastically to any analysis period, which is done automatically by the computational engine;
- The incident duration is assigned stochastically on the basis of the severity-defined incident duration distribution;
- The incident location is assigned stochastically, weighted by the individual segment VMT; and
- The incident severity is assigned stochastically on the basis of the distribution of incident severity.

Default adjustment factors for incidents are provided in Section 5.

Work Zones

This portion of the analysis pertains exclusively to scheduled, significant work zone events. Minor patching and repair activities are not treated as work zones, but these important activities can be treated as incident events in the procedure and may be added to the incident tally. Thus, a work zone constitutes any activity that results in scheduled closures of the shoulder or one or more travel lanes. Typically, a work zone lasts multiple days or weeks. In some cases, it involves multiple stages, each with different shoulder- and lane-closure parameters.

The details of scheduled work zone activities must be entered by the analyst and cannot be defaulted. A work zone log should be entered in which the following information is input for each work zone activity that is planned during the RRP:

- Calendar days of the start and end dates of the work zone activity,
- Facility segment(s) and time periods affected by the work zone activity,
- Portions of the facility cross section affected by closures (i.e., shoulder, one-lane, or multiple-lane closures),
- Type of barrier used to separate traffic from the work activity (i.e., concrete or other hard barrier; cones, drums, or other soft barrier),
- Regulatory speed limit in effect during the work activity, and
- Lateral separation between traffic and the work zone.

The methodology can accommodate multiple work zone activities, each with its own sets of inputs. Capacity adjustment factors (CAFs) and speed adjustment factors (SAFs) for work zones have been developed by national research (*3, 4*) and are described in Section 4 of Chapter 10, Freeway Facilities Core Methodology.

A schematic illustration of the time–space domain for a scenario containing weather and incident events is shown in Exhibit 11-5. The freeway facility in question consists of 10 analysis segments and is analyzed over a 3-h study period (12 analysis periods). This scenario contains a rain event (R) that starts 45 min into the study period and lasts for 45 min. Weather is assumed to affect the entire facility equally.

Analysis	Segment Number									
Period	1	2	3	4	5	6	7	8	9	10
1										
2										
3	R	R	R	R	R	R	R	R	R	R
4	R	R	R	R	R	R	R	R	R	R
5	R	R	R	R	R	R	R	R and I-2	R	R
6								I-2		
7								I-2		
8								I-2		
9										
10										
11			I-S							
12										

Exhibit 11-5
Scenario Illustrating Weather and Incident Events

Exhibit 11-5 also shows an incident blocking two lanes of Segment 8 (I-2) starting 75 min into the study period. This incident is concurrent with the rain event in Analysis Period 5, and the incident duration is 1 h. Another minor incident (I-S) closes the shoulder of Segment 3 in Analysis Period 11. All shaded cells in Exhibit 11-5 (i.e., combinations of analysis segment and analysis period) will experience some reduction in capacity and possible changes in free-flow speed and traffic demand.

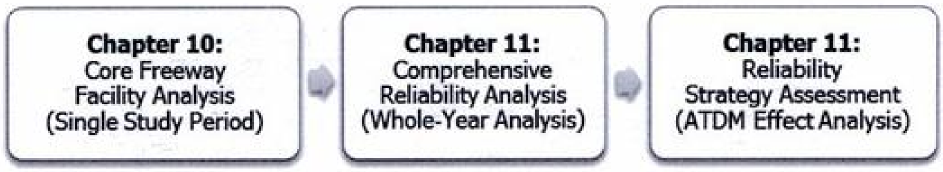
When two independent events affect capacity at the same time, their combined effect is the multiplication of the two CAFs. This would be the case for Segment 8 in Analysis Period 5, where the product of the rain event CAF and the incident event CAF would be applied. This is also true if the example had included a work zone event, which would have likely affected the CAFs and SAFs for all analysis periods on any segments having work activity.

ACTIVE TRAFFIC AND DEMAND MANAGEMENT

ATDM concepts for freeway facilities are presented in Chapter 37, ATDM: Supplemental. The concepts presented below pertain to how ATDM is integrated into the freeway facilities core and reliability methodologies. The ATDM methodology was initially developed by Federal Highway Administration (FHWA) research (5) and has been adapted to fit within the HCM's scenario generation and reliability evaluation methodology.

The ATDM methodology requires the analyst to carry out the freeway facilities core and reliability analyses before testing any ATDM strategies, as illustrated in Exhibit 11-6. This sequence is required because many ATDM strategies are targeted to mitigate the impacts of specific types of recurring or nonrecurring events. For example, if incident-induced delays are significant, a strategy could be to deploy or increase the frequency of freeway service patrols to reduce the capacity impacts of those incidents. Obviously, this strategy will apply only to scenarios where incidents occur. On the other hand, a recurring bottleneck at a freeway on-ramp could be mitigated by implementing a ramp-metering strategy across the whole year. In summary, *any subset* of the reliability scenarios can be viewed as the "before" case for ATDM analysis, while the scenarios selected for mitigation via ATDM can be viewed as the "after" case.

Exhibit 11-6
Process Flow for ATDM Implementation for Freeway Facilities



Three types of comparisons are provided in the procedure to quantify the effects of ATDM strategies on freeway facility operations.

- The first comparison is done at the *individual scenario level*. It allows the effects of specific events and strategies to be evaluated and can be used as the basis for large-scale ATDM analyses later. This type of comparison can be used to judge the relative effects of different ATDM strategies *on a common scenario* to aid the decision-making process.
- The second comparison makes use of *all scenarios selected by the analyst* in the "after" ATDM subset and evaluates performance changes between the collection of multiple "before" and "after" sets. This comparison considers only the scenarios that are included in the ATDM "after" set and does not consider any other scenarios. For example, if an "after" ATDM set is applied to 25 scenarios, the second-level comparison will consider the "before" and "after" ATDM outputs only for those 25 scenarios. This comparison *does not provide any insights into ATDM impacts on reliability.*
- The final comparison extrapolates the effects of the ATDM analysis to the *entire set* of all reliability scenarios and seeks to answer the following question: How do ATDM strategies applied to a *selected* number of scenarios affect reliability performance measures over the full RRP? Here, the distribution of performance measures for the entire reliability analysis can be compared with that of the ATDM "after" set, which effectively treats the reliability scenarios as the "before" case. For the "after" case, which contains some scenarios that include ATDM strategies, adjustments to the scenario probabilities are made to match the original TTI distribution of the set of reliability scenarios, a process described in detail in Chapter 25. Once this adjustment has been completed, the distributions of performance parameters and other outputs can be compared and conclusions formed about the effectiveness of the ATDM strategies.

3. METHODOLOGY

This section describes the methodology for evaluating the travel time reliability of a freeway facility. It also describes extensions to the freeway facilities core methodology (Chapter 10) that are required for computing reliability performance measures.

The freeway reliability methodology is computationally intense and requires software to implement. The intensity stems from the need to create and process the input and output data associated with the hundreds of scenarios considered for a typical RRP. The objective of this section is to introduce the analyst to the calculation process and to discuss the key analytic procedures. Important equations, concepts, and interpretations are highlighted. The computational details of the methodology are provided in Chapter 25, Freeway Facilities: Supplemental.

SCOPE OF THE METHODOLOGY

Framework

The freeway reliability methodology includes a base dataset, the scenario generator, and the core computational procedure from Chapter 10. The computational procedure predicts travel times for each analysis period in each scenario. They are subsequently assembled into a travel time distribution that is used to determine performance measures of interest. These components are illustrated in Exhibit 11-7.

Exhibit 11-7
Freeway Reliability Methodology Framework

Exhibit 11-8 provides an overview of the reliability parameters for geometry, demand, weather, work zones, and incident events. It describes how these parameters are treated in the three parts of the scenario generation process: (*a*) treated deterministically in the base scenario (Chapter 10), (*b*) treated deterministically in scenario generation (Chapter 11), or (*c*) treated stochastically in scenario generation.

Exhibit 11-8
Overview of Reliability Parameters

Reliability Parameter		Treated Deterministically in Seed File	Treated Deterministically in Scenario Generation	Treated Stochastically in Scenario Generation
Facility geometry		Segmentation, number of lanes, free-flow speed, etc.	NA	NA
Traffic demand level		15-min flow rates represent 1 day in base scenario	Variable based on day of week and month of year	NA
Weather events	Duration	NA	User input or default values	NA
	Start time	NA	NA	Stochastically assigned to analysis periods
	Frequency	NA	User input or default values	Stochastically assigned to scenarios
Work zones	Long term (entire RRP)	Work zone duration, segments, schedule in base scenario	NA	NA
	Short term (less than RRP)	NA	User input in specific scenarios	NA
Incident events	Duration	NA	User input or default values	Stochastically determined on the basis of user inputs
	Start time	NA	NA	Stochastically assigned to analysis periods
	Location	NA	NA	Stochastically assigned to segments
	Frequency	NA	User input or default values	Stochastically assigned to scenarios
	Severity	NA	User input or default values	Stochastically assigned to scenarios

Note: NA = not applicable.

Base Dataset

The base dataset contains all the required inputs for executing both the freeway facility core methodology and the reliability analysis.

The base dataset provides all the required input data for the freeway facilities core methodology described in Chapter 10. Some data are specific to the freeway facility being studied. These items include, at a minimum, all segment geometries (both general purpose and managed lanes, if applicable), free-flow speeds, lane patterns, and segment types, along with base demands that are typically, but not necessarily, reflective of average [annual average daily traffic (AADT)] conditions. In addition, the base dataset contains the input data required for executing this chapter's reliability methodology. These data include a demand multiplier matrix, weather, work zones, and incident events as described later in this section. Most of the reliability-specific input data can be defaulted when they are not available locally, but the analyst is encouraged to supply facility-specific data whenever feasible.

Scenario Generation

The scenario generator develops a set of scenarios reflecting conditions that the freeway facility may experience during the RRP. Each scenario represents a single study period (typically several hours long) that is fully characterized in terms of demand and capacity variations in time and space. The data supplied to the scenario generator are expressed as multiplicative factors [CAFs, SAFs, and demand adjustment factors (DAFs)] or additive factors (number of lanes) that are applied to the base free-flow speed, demand, capacity, and number of lanes.

The scenario generation process includes the following steps:

- Adjusting the base demand to reflect day-of-week and month-of-year variations associated with a given scenario;
- Generating inclement weather events on the basis of their local probability of occurrence in a given time of year and adjusting capacities and free-flow speeds to reflect the effects of the weather events;
- Generating various types of incidents on the basis of their probability of occurrence and adjusting capacities to reflect their effects; and
- Incorporating analyst-supplied information about when and where work zones and special events occur, along with any corresponding changes in the base demand or geometry.

The results from these steps are used to develop one scenario for each study period in the RRP.

Facility Evaluation

In the facility evaluation step, each scenario is analyzed with the freeway facilities core methodology. The performance measures of interest to the evaluation—in particular, facility travel time—are calculated for each analysis period in each scenario and stored. At the end of this process, a travel time distribution is formed from the travel time results stored for each scenario.

Performance Summary

In the final step, travel time reliability is described for the entire RRP. The travel time distribution is used to quantify a range of variability and reliability metrics.

Spatial and Temporal Limits

The reliability methodology is subject to the same spatial and temporal limits as the freeway facilities core methodology. The RRP can be as long as 1 calendar year, although shorter periods are possible. A 1-year RRP is most typical, since it encompasses all day-to-day and month-to-month variability in demand, as well as all weather and incident effects. However, shorter RRPs can be used to focus on reliability during specific time periods. The minimum recommended RRP is 1 month to capture sufficient variability in demand and other factors.

For a 1-year RRP, the methodology is typically applied with four replications for each of 5 weekdays (Monday through Friday) and 12 months in the year, for a total of 240 scenarios. This approach roughly corresponds to 250 work days in a typical calendar year. A reliability analysis that includes weekend effects would result in an increased number of scenarios.

For RRPs that are significantly shorter than 1 year, the analyst should increase the number of replications to ensure a sufficient sample size for scenario generation. Exhibit 11-9 provides guidance on the recommended number of replications in such cases.

Exhibit 11-9
Recommended Number of Replications for Scenario Generation

RRP Duration (months)	Number of Days Considered	Recommended Number of Replications	Resulting Number of Scenarios
1	5 (all weekdays)	48	240
2	5	24	240
4	5	12	240
6	5	8	240
9	5	6	270
12[a]	5	4[a]	240[a]
12	2 (weekend only)	10	240
12	7 (all days)[b]	3	252

Notes: RRP = reliability reporting period.
[a] Default value.
[b] Not desirable; separation of weekday and weekend reliability analysis is preferred.

For the base scenario provided as part of the base dataset, there is no limit to the number of time periods that can be analyzed. The computational engine supports an evaluation of a 24-h period. The duration of the study period should be sufficiently long to contain the formation and dissipation of all queues. The facility length evaluated should be less than the distance a vehicle traveling at the average speed can travel in 15 min. This specification generally results in a maximum facility length between 9 and 12 mi. Longer facilities may be evaluated, but results need to be interpreted carefully, since the onset of congestion in later time periods may be estimated to occur earlier than field observations would indicate. More discussion on facility length is provided in Chapter 10.

Performance Measures

There are many possible performance measures for quantifying aspects of the travel time reliability distribution. The following measures, defined in Section 2 of Chapter 4, Traffic Operations and Capacity Concepts, are among the more useful for quantifying differences in reliability between facilities and for evaluating alternatives to improve reliability. All of these measures are produced by the freeway travel time reliability methodology:

- TTI_{95} (i.e., PTI) (unitless),
- TTI_{80} (unitless),
- TTI_{50} (i.e., median TTI) (unitless),
- TTI_{mean},
- Failure and on-time measures (percentage),

- Reliability rating (percentage),
- Semi–standard deviation (unitless),
- Standard deviation (unitless), and
- Misery index (unitless).

In addition, all of the performance measures generated by the freeway facilities core methodology (Chapter 10) are computed for each general purpose and managed lane segment for each analysis period being evaluated.

Strengths of the Methodology

The methodology is capable of estimating the impacts of nonrecurring congestion (due to demand variability, weather, incidents, work zones, and special events) on the operational performance of a freeway facility over an extended RRP—up to 1 year. Because of the computational efficiency of the HCM freeway facilities core methodology compared with, for example, a simulation analysis of a freeway facility, a whole-year analysis can be performed relatively quickly. The following are specific strengths of the methodology:

- It is an efficient method for estimating travel time reliability. It can be applied quickly several hundred times to derive a travel time distribution over RRPs of up to 1 year.
- The core methodology is less computationally intensive than microsimulation.
- The core methodology can be directly calibrated on the basis of local or regional capacity defaults to replicate recurring bottlenecks.
- It considers local and regional weather defaults for the 101 largest U.S. metropolitan areas on the basis of a 10-year average.
- It encompasses a method for estimating incident and crash rates in the absence of detailed local incident logs.
- The method can be extended to evaluate ATDM strategies.

In addition, the strengths of the core methodology described in Chapter 10 apply to the reliability and strategy assessment methods presented here.

Limitations of the Methodology

Because the reliability method applies the freeway facilities core methodology multiple times, it inherits the core methodology's limitations. These limitations were described in Chapter 10. For example, one limitation of the core method is its use of 15-min analysis periods. Therefore, all event durations (e.g., weather, incidents) used by the reliability method must be expressed as integer numbers of 15-min analysis periods. The reliability method has the following additional limitations:

- The method assumes that the effect of two or more factors (weather and incident) on speed or capacity is multiplicative. This assumption has not been sufficiently tested empirically and may overstate the influence of combined nonrecurring congestion effects.

- Weather events with a small capacity reduction effect (<4%) are not addressed. A given weather event (e.g., rain, snow) is always assumed to occur at its mean duration value. Sun glare is not accounted for.
- The method assumes that incident occurrence and traffic demand are independent of weather conditions, although all are indirectly tied to each other through the specification of demand, incident, and weather probabilities on a calendar basis. However, the analyst is able to adjust incident frequencies by month on the basis of local data.
- The method estimates incident occurrence as a function of segment demand and month of the year. It does not consider potentially elevated incident rates in segments with low demands. Some segments may be overly prone to incidents due to poor visibility, poor geometry, a short weaving segment, or other factors that are not considered by the reliability method.
- The method does not consider full facility closures in the scenarios. In assigning incidents to the segments, at least one lane should therefore remain open. The scenario generation methodology does not assign incidents that result in full segment closure; it reassigns those probabilities to other (less severe) incidents. This is also true for work zones, where at least one travel lane has to remain open.
- The travel time reliability analysis assumes similar effects of demand variation and weather conditions on general purpose and managed lanes, when a managed lane facility is included in the analysis.
- Work zone events are only allowed to be modeled in general purpose lanes; no managed lane work zone effects are considered.
- The traffic demand adjustment assumes a proportional demand effect across the entire facility, which means that all inputs and outputs (across time and space in the base scenario) are increased or decreased by the same factor.

REQUIRED DATA AND SOURCES

As a starting point, all of the input data normally needed in applying the freeway facilities core methodology are required. These requirements are given in Chapter 10. A base scenario is always required and is used to describe base conditions (particularly demand and factors influencing capacity and free-flow speed). The base scenario is intended to represent average demand conditions (e.g., AADT) or the demand measured on a specific day. This chapter's methods factor these demands on the basis of user-supplied or defaulted demand patterns to generate demands representative of all other time periods during the RRP.

Additional data beyond those necessary for an HCM freeway facility analysis are required for a reliability evaluation. Exhibit 11-10 lists the general categories of data that are required by data type. Details are provided in the following subsections.

Exhibit 11-10
Required Input Data, Potential Data Sources, and Default Values for Freeway Reliability Analysis

Data Category	Potential Data Source	Suggested Default Value
Time periods	User-defined study period, representative data of base scenario, and RRP	Must be provided
Demand multipliers	Field data or modeling to generate day-of-week by month-of-year demand factors	Urban and rural defaults provided in Section 5
Weather	Online database for probabilities of various intensities of rain, snow, cold, and low visibility by month	Defaults for 101 largest U.S. metropolitan areas provided in Chapter 25
Incidents	Field data estimates of frequencies of occurrence of shoulder and lane closures per study period for each month, incident severity distribution, and average incident durations; alternatively, crash rate and incident-to-crash ratio for the facility, in combination with defaulted incident type probability and duration data	Estimated from segment AADT and lengths as described in Chapter 25
Work zones and special events	User input on changes to base conditions and their schedule	Must be provided
Nearest city	Select from the list of metropolitan areas provided in the Volume 4 Technical Reference Library	Must be provided when default weather data are used
Geometrics	No details beyond core methodology needed. Obtained from road inventory or aerial photo	Must be provided
Traffic counts	Demand multiplier represented in base dataset. Base scenario data from field data or modeling	Must be provided

As shown in Exhibit 11-10, most reliability-specific inputs can be defaulted or are already required by the core methodology. Section 5, Applications, provides default values that allow analysts in "data poor" regions lacking detailed demand, weather, or incident data to apply this chapter's methods and obtain reasonable results. At the same time, the method allows analysts in "data rich" regions to provide detailed local data for these inputs when the most accurate results are desired.

Although default values are provided for many of the variables that affect facility reliability (see Section 5, Applications), travel time reliability (as measured by TTI_{80} or TTI_{95}) can vary widely, depending on the characteristics of a particular facility and the length of the study period. Therefore, analysts are encouraged to use local values representative of local demand, weather, and incident patterns whenever such data are available. In addition, analysts must supply local values for work zones and special events if they wish to account for these effects in a reliability analysis. This subsection identifies potential sources of these data.

Demand Patterns

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

Weather

The National Climatic Data Center (NCDC) provides rainfall, snow, and temperature statistics for thousands of locations through its website (*6*) and average precipitation rate data in the *Rainfall Frequency Atlas* (*7*). The more detailed hourly weather data needed for a freeway facility analysis are available from larger airport weather stations and can be obtained from the NCDC website or other online sources [e.g., Weather Underground (*8*)].

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

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

Incidents

A significant level of effort is required to extract information about the numbers and average durations of each incident type from the annual incident logs maintained by roadway agencies, even in data-rich environments. Furthermore, certain incident types—particularly shoulder incidents—can be significantly underreported in incident logs (*1*). Thus, the direct approach of estimating incident probabilities is reserved for the rare cases where incident logs are complete and accurate over the entire RRP. An alternative approach is to estimate the facility incident rate from its predicted crash rate and assume that the number of incidents in a given study period is Poisson distributed (*9, 10*). Details of the process are described in Chapter 25, Freeway Facilities: Supplemental.

Work Zones

A schedule of long-term work zones indicating the days and times when the work zone will be in force and the portions of the roadway that will be affected should be obtained from the roadway operating agency. Work zones that vary in intensity (e.g., one lane closed on some days and two lanes closed on others) or that affect different segments at different times will need to be specified as two different work zones. When detailed traffic control plans for each work zone are available, they should be consulted to determine the starting and ending locations of lane closures, along with any reductions in the posted speed. When detailed plans are not available, the agency's standard practices for work zone traffic control can be consulted to determine the likely traffic control that would be implemented, given the project's characteristics.

Special Events

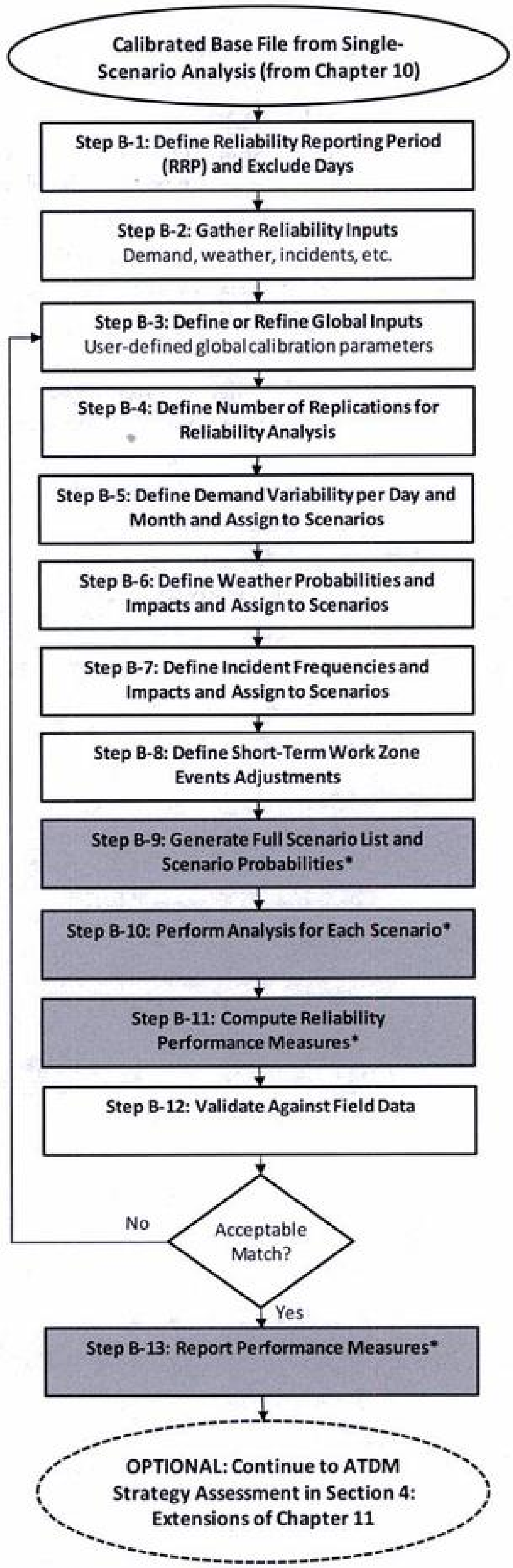
Special events are short-term events, such as major sporting events, concerts, and festivals, that produce intense traffic demands on a facility for limited periods. Special traffic control procedures may need to be implemented to accommodate the traffic demands. The analyst should identify whether any events that occur in or near the study area warrant special treatment. If so, a schedule for the event (dates, starting times, typical duration) should be obtained. Some types of events also have varying intensities that will require separate treatment (e.g., a sold-out baseball game compared with a lower-attendance midweek game). Recurring events may have developed special traffic control procedures; if so, these plans should be consulted to identify any changes required from base conditions. Each combination of special event venue and event intensity to be included in the analysis will need to be specified.

METHODOLOGY OVERVIEW

The methodologies for freeway reliability and freeway strategy assessment are the second and third of the three parts of an evaluation sequence starting with the evaluation of the freeway facility for a base scenario. Part A: Core Freeway Facility Analysis (Single Study Period) was presented in Chapter 10. Part B: Comprehensive Freeway Reliability Analysis is the methodology presented in this section. Part C: Reliability Strategy Assessment is presented in Section 4. It allows for the evaluation of ATDM strategies.

Completion of the core methodology's computational steps (Steps A-1 through A-17) is a prerequisite for conducting a reliability analysis (Steps B-1 through B-13, depicted in Exhibit 11-11). Completion of a reliability analysis is a prerequisite for an ATDM strategy assessment (Steps C-1 through C-9, presented in Section 4).

Exhibit 11-11
Freeway Reliability Methodology Framework



Note: * Steps shaded in gray are performed by the computational engine.

COMPUTATIONAL STEPS

This section describes the reliability methodology's computational steps. To simplify the presentation, the focus is on the function of and rationale for each step. Chapter 25, Freeway Facilities: Supplemental, contains an expanded version of this section that provides the supporting analytical models and equations.

Step B-1: Define RRP and Exclude Days

In this step the analyst defines the duration of the RRP, which is typically 1 calendar year to encompass all day-to-day and month-to-month variability in demand as well as all incident and weather patterns observed over the calendar year. Periods shorter than 1 year can be selected for specific analysis questions, in 1-day increments. For example, an analyst may be interested in evaluating the reliability of a freeway only during the summer tourist season or during the local construction season (excluding winter months). In combination with the strategy assessment extensions described in Section 4, an analyst may decide to evaluate a weather management program and impacts of freeway service patrols only for the winter months. As described earlier, selecting a shorter RRP will generally require more replications of the scenario generation process. RRPs longer than 1 year are not recommended, because all variability sources considered in the method are captured in a 1-year duration.

In this step the analyst also decides which days of the week to include in the analysis. A reliability analysis is typically performed for the 5 weekdays, although weekends can be included if desired. Exhibit 11-9 provided guidance on the number of replications recommended for a weekend-only analysis. However, if a facility experiences significantly different performance on weekdays and on weekends or if different weekday and weekend driver populations (e.g., commuter versus recreational trips) are known to exist, the mixing of weekdays and weekends in the same reliability analysis is strongly discouraged.

In defining the RRP, the analyst may decide to exclude 1 or more days from the analysis. If the analyst is interested in "typical" weekday performance, the analyst may wish to exclude holidays (and high-demand travel days before or after the holiday itself) from the analysis.

The reliability analysis works from a base scenario that is evaluated with the freeway facility core method presented in Chapter 10. Because the methodology adjusts seasonal and day-of-week demand patterns relative to the base scenario, the specific date represented by the base scenario needs to be defined. That is, the demand values contained in the base scenario should correspond to a specific day–month combination of the year; these demands are then adjusted by the reliability method for the other scenarios it generates.

Alternatively, the analyst may choose to provide demands representative of an "average day" in the base scenario on the basis of AADT values. In this case, the analyst would then use demand multipliers in Step B-5 that are calculated relative to that average day, rather than to the base scenario day. In other words, Steps B-1 and B-5 need to be coordinated to ensure that the correct demand multiplier factors are applied.

Step B-2: Gather Reliability Inputs

This step collects the additional inputs needed for conducting a reliability analysis, including demand variability by day of week and month of year, weather data, incident records, work zone data, and special events. Some default values and quick estimation methods are provided to aid the analyst; these are described in detail in Chapter 25. A list of required data and potential data sources was presented above.

Step B-3: Define or Refine Global Inputs

In this step, the analyst has a chance to revise two global calibration parameters for the analysis: facilitywide jam density and the queue discharge capacity drop. While multiple bottlenecks (with different CAFs) can exist along a facility, these two parameters are assumed to be global for the entire facility. This step should be treated with care, since these two parameters were previously defined and calibrated for the core facility analysis. While these parameters provide additional calibration tools for reliability analysis, having a well-calibrated base file is preferable, and changing global inputs for reliability assessment is not generally recommended. In general, the output of a reliability analysis is better calibrated by varying DAFs, CAFs, SAFs, the number of lanes closed by incident types, and the underlying scenario probabilities. A detailed reliability calibration methodology is presented in Chapter 25.

Step B-4: Define Number of Replications for Reliability Analysis

In this step, the analyst specifies the number of replications used to generate scenarios. The default number of replications for a 12-month RRP is four, to ensure a sufficiently large sample of randomly generated weather and incident events. The Spatial and Temporal Limits discussion earlier in this section, along with Exhibit 11-9, provides guidance for modifying the number of replications for shorter RRPs.

The goal of the hybrid scenario generation approach (with some deterministic and some stochastic inputs) is to reduce the number of scenarios from potentially several thousand to a few hundred representative scenarios that capture the effects of all sources of nonrecurring congestion. For most reliability applications, 240 scenarios (5 weekdays, 12 months, and 4 replications) are sufficient to capture the 1-year variability in performance. However, the analyst may choose to include rarer scenarios (e.g., a 5- or 10-year storm) to evaluate the impacts of very rare events. When an ATDM strategy evaluation will also be conducted, a smaller number of scenarios is recommended to allow for scenario-specific selection of ATDM strategies, as discussed in Section 4.

Step B-5: Define Demand Variability by Day and Month and Assign to Scenarios

This step defines demand multipliers by day of the week and by month of the year on the basis of facility-specific data. The demand multiplier is expressed relative to the base scenario demand date from the core freeway facility defined in Step B-1. Alternatively, the analyst may select an average demand day (estimated from AADTs) and express demand variability relative to that day.

The base scenario day does not need to be an average day (i.e., it can have high or low demand relative to average conditions, which is accounted for in this step) but should be free of special events or nonrecurring sources of congestion.

Default values for urban and rural demand patterns are provided in Section 5. They were developed from a national freeway demand dataset (2). However, facility-specific data are strongly preferred and are usually readily obtainable from permanent traffic count stations or online sensor databases.

Step B-6: Define Weather Probabilities and Impacts and Assign to Scenarios

This step defines the probabilities of occurrence of each of the HCM weather types, along with corresponding CAFs, SAFs, and DAFs. They are timewise probabilities. They represent the chance of occurrence of a weather event at any instant in time and do not correspond to frequencies of weather events. In other words, frequencies of weather events are converted to probabilities on the basis of time of day and month of year. Default weather type probabilities are provided for the 101 largest U.S. metropolitan areas in the Volume 4 Technical Reference Library. CAF and SAF defaults are provided in Section 5, but values developed from local data can be used instead.

No default DAFs are available at this time, although extreme weather events are generally understood to affect traffic demands. For example, nighttime or early morning snowstorms are expected to reduce the demand levels in the a.m. peak period, while multiday snow events are likely to reduce both a.m. and p.m. peak demands. This effect also depends on location (e.g., Boston versus Atlanta). Afternoon snowstorms may be less likely to affect p.m. peak demand, since commuters may not have altered their home-to-work trips that morning. Analysts are encouraged to develop customized weather demand adjustment factors or apply judgment on the basis of local conditions and experience.

Step B-7: Define Incident Frequencies and Impacts and Assign to Scenarios

This step defines incident frequencies for each of the HCM incident severity types, along with the corresponding CAFs, SAFs, and DAFs and the number of lanes lost due to the incident. Default CAF and SAF values are provided in Section 5, while DAFs will need to be user-defined. A quick method for estimating incident frequencies on the basis of each segment's daily demand levels is provided in Chapter 25. However, facility-specific data are preferable in specifying incident frequencies.

Chapter 25's incident frequency estimation considers the total traffic demand on a segment on the day represented by the base scenario to generate incident frequencies for reliability analysis. Because different analysis segments have different demand levels, the estimated incident rates will also differ as a function of that demand. Accordingly, the scenario generation step is more likely to generate more incidents on segments with higher demand, which affects the overall reliability performance.

User-specified incident rates are especially important if an analyst is aware of recurring monthly variations in incidents. If, for example, incidents are more likely in winter months (despite potentially lower demands), the analyst should adjust the incident rate defaults and calibrate for local conditions.

Step B-8: Define Short-Term Work Zone Events and Adjustments

This step defines the dates of any short-term work zone events, along with the corresponding CAFs, SAFs, and DAFs and the number of lanes lost due to the work zone. The phrase "short-term work zones" in this case refers to scheduled or planned work zones that do not cover the entire RRP. For example, if a work zone is in place for 1 or 2 months in a 1-year RRP, the configuration should be entered here.

Long-term work zones, or those that cover the entire RRP, should be evaluated as a stand-alone reliability analysis, with a base scenario modified to reflect the work zone characteristics. One exception is a long-term work zone that covers the entire RRP but that is divided into different stages or configurations with varying CAFs, SAFs, or DAFs or different affected segments. In that case, each stage can be accounted for separately and sequentially in this step.

DAFs for short-term work zones are user-defined. A method for estimating CAF and SAF values for work zones is provided in Section 4 of Chapter 10.

Nonscheduled work zones, including very short (i.e., single-day) activities (e.g., shoulder closure for landscaping work, lane closure for pothole filling), are best addressed as a form of (random) incident in Step B-7 rather than by explicitly defining their occurrence and location in this step.

Step B-9: Generate Full Scenario List and Scenario Probabilities

This step generates the listing of all scenarios for reliability analysis on the basis of the inputs provided in the previous steps. The step is automatically executed by the computational engine or other software tools. The number of scenarios is a function of the user input in previous steps, including the length of RRP (in months), the number of days generally included in each week, the number of days specifically excluded, and the number of replications. The scenario generation process is summarized here and described in detail in Chapter 25, Freeway Facilities: Supplemental.

Each scenario will have a complete set of attributes defining the characteristics of that scenario relative to the base scenario. Specifically, each scenario will have a series of five matrices that define the demand multipliers (from Step B-5), along with CAF, SAF, and DAF values and adjustments to the number of lanes (Steps B-6 through B-8). The size of each of the adjustment matrices will be equal to the number of analysis segments times the number analysis periods contained within the base scenario. When managed lanes are included in the analysis, the size of these matrices will double to provide similar information for the managed lanes.

Whenever a scenario contains multiple adjustment effects due to weather, incidents, or work zones, the methodology assumes that any two or more CAFs, SAFs, or DAFs are multiplicative (i.e., independent). The number-of-lanes adjustment factors are additive for incident and work zone events.

For example, with regard to the weather and incident combination in Exhibit 11-5, the size of each adjustment matrix is 10 segments by 12 time periods. All 120 cells will be subject to demand multipliers from Step B-5. In addition, a 45-min rain event in Analysis Periods 3 through 5 will result in CAF, SAF, and DAF adjustments for the entire 10-segment facility during those time periods (30 cells). A two-lane closure incident in Segment 8 in Analysis Periods 5 through 8 will reduce the number of lanes in that segment for those four time periods. In addition, CAF, SAF, and DAF adjustments are provided. The incident overlaps the rain event in one of the time periods (Segment 8, Analysis Period 5), resulting in a multiplicative effect of adjustments due to weather and incident. Finally, a 15-min shoulder-closure incident in Segment 3 in Analysis Period 11 results in CAF, SAF, and possibly DAF adjustments.

If 240 scenarios are generated for the example in Exhibit 11-5, a total of 144,000 (5 adjustment matrices × 120 cells per matrix × 240 scenarios) adjustment factors will be applied. The computational engine or other software automatically performs the record keeping and estimation of these factors.

Step B-10: Perform Analysis for Each Scenario

This step automatically processes each scenario in the computational engine or other software. The adjustment matrices from Step B-9 are applied sequentially to the base scenario, and the resulting scenarios are evaluated individually with the Chapter 10 core methodology. The computational engine or software produces the facilitywide performance measures for each scenario.

Step B-11: Compute Reliability Performance Measures

This step generates a travel time distribution from the stored average facility travel times by analysis period and scenario. It also computes a variety of reliability performance measures from the results of all scenarios:

- TTI_{95} (PTI),
- TTI_{80},
- TTI_{50},
- TTI_{mean},
- Reliability rating,
- Semi–standard deviation,
- Standard deviation,
- Failure or on-time percentage based on a target speed,
- Policy index based on a target speed, and
- Misery index.

These performance measures were defined in Section 2. Their computation is automated by the computational engine or other software. Additional details for computing reliability performance measures are provided in Chapter 25.

The example facility shown in Exhibit 11-5 will generate 12 facility travel times per scenario, one per analysis period. Multiplication by 240 scenarios will result in 2,880 facility travel time observations that define the full travel time distribution. When these observations are sorted from highest to lowest, the TTI_{95} is the travel time value ranked number 144 (0.05 × 2,880) in the sorted list, while the TTI_{50} is the value ranked 1,440, and so on.

Step B-12: Validate Against Field Data

In this step, the reliability results are compared with field data, results from another model, or expert judgment if no other data are available. If an acceptable match is not obtained, the analysis returns to Step B-3 to make calibration adjustments and then repeats the subsequent steps. Additional details on criteria for calibrating and validating the facility are presented in Chapter 25.

Step B-13: Report Performance Measures

This final step of the reliability assessment methodology reports the facility's reliability performance measures. Step B-13 concludes the reliability analysis methodology. At this time, the analyst may choose to continue to perform an ATDM evaluation, as described in Section 4. Note that no level of service is defined for a reliability analysis. The analysis instead presents various reliability performance measures, as well as the resulting travel time distribution.

4. EXTENSIONS TO THE METHODOLOGY

ACTIVE TRAFFIC AND DEMAND MANAGEMENT

ATDM is the dynamic management, control, and influence of travel demand, traffic demand, and traffic flow on transportation facilities. Through the use of tools and countermeasure strategies, traffic flow is managed and traveler behavior is influenced in real time to achieve operational objectives. The objectives include preventing or delaying breakdown conditions, improving safety, promoting sustainable travel modes, reducing emissions, and maximizing system efficiency.

This section provides an analysis framework, recommended measures of effectiveness, and a methodology for evaluating the impacts of ATDM strategies on freeway demand, capacity, and performance. Although this section describes various ATDM "strategies" and "measures," almost any system management or operations strategy that is applied in a dynamic manner can be considered active management.

The methodology presented here is primarily focused on traffic management applications. In some cases, the operational strategies presented here may be relatively static (e.g., fixed ramp-metering rates or pricing schedules). The primary focus of ATDM analysis in the HCM is to provide practitioners with practical, cost-effective methods for representing the varied demand and capacity conditions that freeway facilities may be expected to operate under. The method enables an analyst to apply a realistic set of transportation management actions to respond to those conditions and thus represent, in a macroscopic sense, the dynamic aspects of ATDM.

The ATDM analysis builds on the freeway reliability analysis methodology, which accounts for freeway performance under different demand, weather, incident, and work zone conditions. The ATDM extension then superimposes one or more strategies on the completed reliability analysis with the goal of improving reliability and other performance measures. Often, the results of an ATDM strategy evaluation would be compared with those of a more traditional capital improvement program that adds physical capacity to the facility in question.

ATDM Strategies and Plans

ATDM strategies are evolving as technology advances. Typical ATDM strategies can be classified according to their purpose and the manner in which they are applied. Among them are the following:

- Ramp-metering strategies,
- Traveler information strategies,
- Managed lane strategies, and
- Speed harmonization strategies.

A more detailed discussion of ATDM strategies is provided in Chapter 37, ATDM: Supplemental. Specialized ATDM programs or plans may be designed to address certain situations. For example, a *weather traffic management plan* may be developed to apply ATDM strategies during adverse weather events. A *traffic incident management plan* may apply ATDM strategies specifically tailored to incidents. A *work zone maintenance-of-traffic plan* may apply ATDM strategies tailored to work zones. *Employer-based demand management plans* may apply major employer–related ATDM strategies to address recurring congestion as well as special weather and incident events.

The ATDM methodology distinguishes between five principal categories of strategies that can affect facility operations:

1. *Demand management strategies* that affect the entire scenario and all segments and analysis periods contained within it when they are invoked through a global increase or (more commonly) a reduction in demand.
2. *Weather management strategies* that influence performance only during analysis periods when a severe weather event affects the facility and apply equally to all segments. Weather management may include driver information, weather-response strategies (e.g., snow removal), and others.
3. *Incident management strategies* that only affect the segment and analysis periods when an incident is present. Incident management may include freeway service patrols that result in reduced incident clearance times, driver information, and others.
4. *Work zone management strategies* that only affect the segment and analysis periods when a work zone is present. Work zone management may include driver information and other strategies.
5. *Special segment-specific strategies* not covered in the previous items, such as hard shoulder running and ramp metering. These strategies specifically alter the capacity of one or more targeted segments and are thus different from global demand management strategies. For example, ramp metering will only affect the entry traffic demand for merge and weave segments. Similarly, hard shoulder running specifically increases capacity in a subset of segments rather than the facility as a whole.

An ATDM plan is a combination of analyst-defined strategies. Conceptually, each ATDM plan combines one or more ATDM strategies into a package of system interventions available to a traffic management center or operating agency. In the context of this methodology, there is no fundamental difference between evaluating a single ATDM strategy and a combination of strategies expressed as a plan. Similar to the reliability methodology described in Section 3, the strategy or plan is ultimately translated into a series of HCM inputs and adjustment factors to demand, capacity, and speed.

From a methodological perspective, only one set of inputs and adjustments can be applied to each reliability scenario. Therefore, if multiple strategies are to be evaluated, they need to be combined into an ATDM plan and then applied to the scenario in question. For example, an incident ATDM plan could include a variable message sign (a demand management strategy) and traffic diversion (an incident management strategy) to avoid or alleviate congestion. The two strategies affect the facility in different ways (since they belong to two different categories) but are combined into a single plan for analysis.

Spatial and Temporal Limits

The ATDM methodology is an extension of the freeway reliability methodology and thus has the same spatial and temporal limits discussed in Section 3.

Limitations of the Methodology

Several limitations apply to the ATDM extensions of this methodology:

- If managed lanes are to be assessed as a strategy in an ATDM analysis, they need to have been included in the base scenario used for the core facility analysis. As described in Chapter 10, Freeway Facilities Core Methodology, managed lanes can affect the segmentation of the facility as well as the scenario generation process. Thus, a "before-and-after" managed lane analysis requires two core facilities, each with a separate reliability analysis.
- This chapter focuses on numerical measures of performance; however, much can be learned by examining graphical measures of performance, such as the facility's speed profile over time and over the length of the facility. This approach can be particularly useful in diagnosing the causes and extent of unreliable performance.
- The ATDM analysis framework translates real-time dynamic control systems into their HCM-equivalent average capacities and speeds for 15-min analysis periods, the smallest unit of time measurement supported by the HCM. Therefore, some of the more dynamic aspects of ATDM must be approximated in this analysis. Because the core methodology for freeway facility analysis is deterministic, only the average impacts of ATDM strategies on demand, speed, and capacity are incorporated in this methodology.
- ATDM is about controlling demand as well as capacity; however, consistent with the rest of the HCM, this chapter focuses on the capacity impacts of ATDM. Demand is an input to these procedures that the analyst must determine with other tools. Demand variability is considered where it influences total demand for the facility (such as peaking within the peak period and variations between days of the year). Demand changes are also considered in the methodology when they are the result of direct controls imposed on the facility, such as ramp metering and vehicle type restrictions (e.g., high-occupancy vehicle lanes and truck lane restrictions). However, prediction of how much additional traffic might be attracted to the facility with the improved performance resulting from ATDM (sometimes called "induced demand") is not included in the chapter's methodology.

Strengths of the Methodology

The following are strengths of the ATDM methodology:

- The ability to target ATDM strategies to scenarios on the basis of their operational characteristics.
- The ability to compare ATDM strategies with traditional, capacity-based facility improvements (e.g., adding lanes).
- The ability to contrast and compare different strategies or sets of strategies in terms of their whole-year effects on the facility. In combination with analyst-supplied cost estimates for the strategies, the method supports a cost–benefit analysis of the strategies.
- The ability to obtain before-and-after comparisons of the effect of ATDM strategies quickly.
- The ability to examine the whole-year effect of specific strategies that may be seasonal (e.g., snow removal) and compare trade-offs with other, nonseasonal strategies.

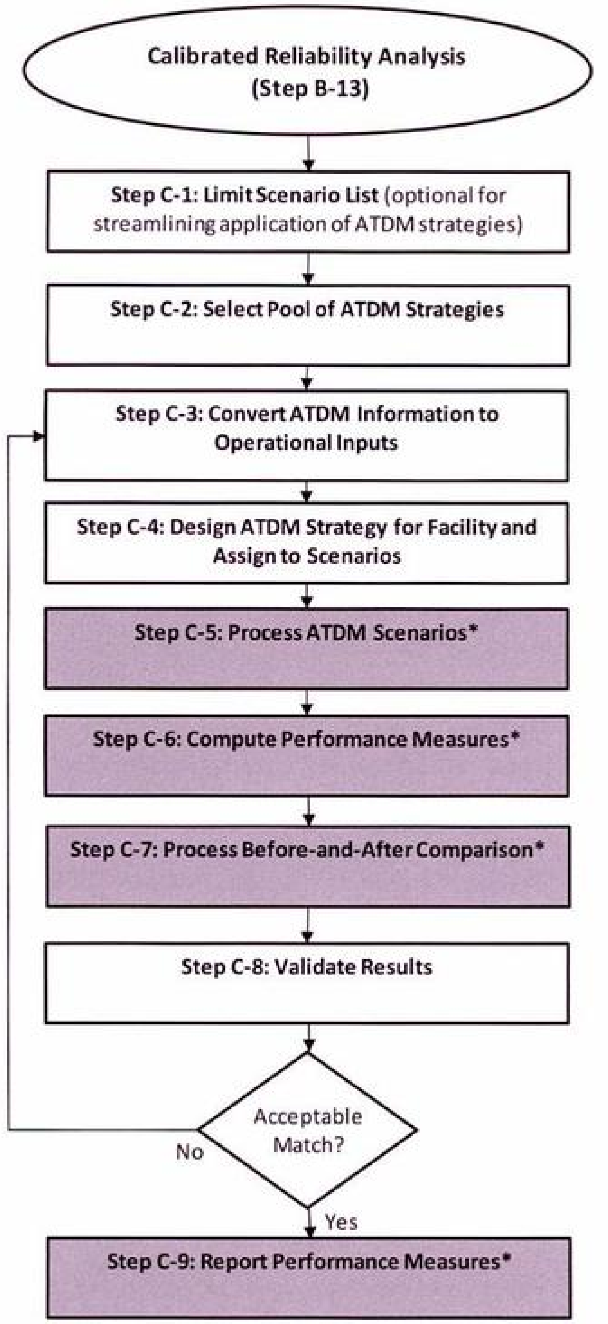
Required Data and Sources

The ATDM methodology requires as input the analyst-defined ATDM strategy or a set of strategies combined into an ATDM plan. The method requires the user to specify the impact of the selected strategies on demand, capacity, free-flow speed, and number of lanes. The impact on demand, capacity, and free-flow speed needs to be converted into matrices of average adjustment factors (DAF, CAF, and SAF) affecting the base condition of the freeway facility in each 15-min analysis period. Guidance and research on the effectiveness of different ATDM strategies are limited.

Adjustments to the Reliability Methodology

The ATDM methodology builds on the reliability analysis described in Section 3, which in turn builds on a calibrated core freeway facility analysis, as described in Chapter 10. The scenarios used for reliability analysis should be generated and calibrated to reflect the facility's operational conditions under different recurring and nonrecurring sources of congestion. Once these steps are taken, the analyst can proceed with the ATDM analysis. Exhibit 11-12 presents the additional nine steps that follow the reliability analysis in performing an ATDM analysis.

Exhibit 11-12
Freeway ATDM Strategy Evaluation Framework



Note: * Steps shaded in gray are performed by the computational engine.

Computational Steps

Step C-1: Limit Scenario List

Testing the effect of ATDM on facility reliability requires a careful selection of the "before" ATDM scenarios.

In this step, the analyst may elect to consider a limited number of scenarios from the reliability analysis to enable a more targeted application of ATDM strategies. The preceding reliability analysis typically results in approximately 240 scenarios for a 1-year RRP. The analyst may apply one or more "global" ATDM strategies equally to all scenarios. In this case, Step C-1 is not necessary and the analysis can proceed.

However, specific ATDM strategies are often applied only to a subset of (reliability) scenarios, to target a specific operational condition. For example, incident management strategies are applied to scenarios with incidents, and work zone management strategies are only applied to scenarios with work zones. At this time, this process of assigning ATDM strategies to scenarios must be carried out manually, since no research results are available to automate the assignment of ATDM strategies to reliability scenarios. To facilitate this process, the list of 240 or so reliability scenarios can be limited to an ATDM subset.

Agencies may have their own algorithms for automating the ATDM strategy assignment process. In that case, no reduction in the number of scenarios is necessary. Similarly, an analyst may elect to assign ATDM strategies manually to all 240 or so scenarios if time and resources permit.

However, in the standard HCM ATDM analysis, the analyst is encouraged to select a subset of scenarios for evaluation. This subset may reflect a certain condition that is targeted by the ATDM strategy in question (e.g., inclement weather days to test a snow removal strategy) but should always include other (nonweather) scenarios, to avoid overestimating the effect of the strategy on the entire RRP. Statistical tests of how well the reduced scenario list reflects the overall population are included in the calibration step.

The framework for ATDM analysis allows the user to select any number or set of reliability scenarios for ATDM or other strategy implementation. However, to generate confidence in the resulting before-and-after comparisons, the analyst should consider the following guidelines for selecting scenarios:

- As general guidance, it is recommended that the analyst select at least 10 scenarios for an ATDM reliability analysis, and preferably 30. Selecting fewer than 10 scenarios may produce significant bias and error in the analysis outputs when the impact on the full system reliability is tested. An ATDM strategy can also be applied to a single scenario in a "before-and-after" core facility analysis by using the method in Chapter 10. Thus, the 10-scenario limit applies only to a *reliability* analysis evaluating before-and-after ATDM effects.
- In comparing the effect of ATDM strategies on the entire set of reliability scenarios, the selection must include broad spectrum scenarios. One or more of these scenarios will need to be a "good operational" scenario, in which the facility travel time is less than the expected value, and one or more of the other scenarios should be a "poor operational scenario." This approach is important for accurate prediction of the impact of the strategy on the full set of reliability scenarios. In other words, the subset of scenarios selected for ATDM analysis should be representative of the overall population of scenarios from the reliability analysis and avoid bias toward overly "good" or "poor" operating conditions. For example, picking a scenario with no inclement weather or incidents has no impact on the results of an "after" scenario when the selected strategy targets improved incident response, but it will nevertheless improve confidence in the comparison of reliability results.

- The selection of ATDM scenarios is best related to the type of strategies that the analyst intends to use. For example, if there is interest in evaluating a set of work zone–related ATDM strategies, the selected scenarios must have some work zone presence.
- If the number of reliability scenarios required for characterizing a certain event (i.e., work zone, weather, incident) is too low to meet the 10-scenario threshold, the analyst should consider increasing the number of replications used in the reliability scenario generation process.

Step C-2: Select Pool of ATDM Strategies

This step allows the analyst to select which ATDM strategy or set of strategies to include in the evaluation. A number of strategies are described in Chapter 37, ATDM: Supplemental. Not every strategy or ATDM plan needs to be applied to every scenario in the ATDM scenario list. For example, a weather management plan may only apply to scenarios with inclement weather, or a freeway service patrol strategy may only apply to incident scenarios.

Step C-3: Convert ATDM Information to Operational Inputs

This step converts the ATDM strategy or plan into operational inputs including DAFs, CAFs, SAFs, incident duration adjustments (if applicable), and number-of-lanes adjustments. The HCM currently does not include default values for ATDM strategies; thus, they must be input by the analyst on the basis of judgment or local data. The reader is referred to Chapter 37, ATDM: Supplemental, for additional information.

Step C-4: Design ATDM Plans for the Facility and Assign to Scenarios

The analyst may elect to apply a strategy uniformly across all scenarios but more commonly would match a specific strategy with a specific scenario (e.g., weather management for snow events, service patrols for incidents).

As discussed earlier, multiple strategies can be combined into an ATDM plan to result in a unique set of inputs (adjustment factors) applied to each scenario. Only one set of these inputs can be applied to each reliability scenario. If multiple strategies are combined, their respective DAFs, CAFs, and SAFs are multiplied to produce a single DAF, CAF, and SAF for application to the scenario, unless additional information is available on the combined effect of pooled strategies.

The computational engine provides the user with a summary sortable table of each scenario's attributes (e.g., number of weather events, number of incidents, maximum TTI) to assist the user in assigning an appropriate set of ATDM strategies to the relevant scenarios.

Step C-5: Process ATDM Scenarios

This step evaluates each scenario by applying the core methodology from Chapter 10. It is automatically performed by the computational engine or other software implementation of the methodology.

Step C-6: Compute Performance Measures

This step calculates performance measures for the facility with the ATDM strategies applied. Results are provided for each scenario, along with an overall travel time (or distribution) using three comparison classes.

The first class compares the performance measure results for a *single scenario* before and after ATDM implementation. This class is useful as an initial test and to verify the scenario assignments carried out in Step C-4. The second class of output compares the *aggregated results* for the combined but limited set of scenarios defined in Step C-1 (e.g., the 30 scenarios selected for ATDM implementation) before and after ATDM implementation. Finally, the third class extrapolates the comparison to the entire travel time distribution across the RRP on the basis of ATDM implementation in a limited set of scenarios.

Step C-7: Process Before-and-After Comparison

This step conducts a before-and-after comparison of ATDM strategy effectiveness by comparing the results of the reliability analysis with the results of the ATDM analysis. The focus of this comparison is on the travel time distribution before and after implementation of the ATDM strategy set. Specific reliability performance measures, including TTI_{mean} and TTI_{95}, can be used for a high-level assessment of the improvement resulting from the ATDM implementation. Generally, though, the overall travel time distribution is of interest in making these comparisons.

Step C-8: Validate Results

In this step, the ATDM results are compared with field data (if available), results from another model, or expert judgment. Field data on the effects of ATDM strategies, especially on the reliability distribution, can be difficult to obtain, and expert judgment may be more frequently applied in this step. Additional details on facility calibration and validation criteria are provided in Chapter 25. If an acceptable match is not obtained, the analysis returns to Step C-3 to adjust the operational inputs.

Step C-9: Report Performance Measures

This final step of the ATDM assessment methodology reports the facility's reliability performance measures with the ATDM strategy or plan applied. Additional performance measures may be generated for each scenario.

5. APPLICATIONS

EXAMPLE PROBLEMS

Section 11 of Chapter 25, Freeway Facilities: Supplemental, provides four example problems that illustrate applications of the reliability and strategy assessment methodologies to a freeway facility under various operating conditions. Exhibit 11-13 lists these example problems.

Exhibit 11-13
List of Example Problems

Example Problem	Description	Application
1	Base reliability	Operational analysis
2	Evaluation of geometric improvements	Operational analysis
3	Evaluation of incident management	Operational analysis
4	Planning-level reliability analysis	Planning analysis

EXAMPLE RESULTS

This section presents the results of applying this chapter's methodologies in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs and to help evaluate whether their analysis results are reasonable. The exhibits in this section are not intended to substitute for an actual analysis and are deliberately provided in a format large enough to depict general trends in the results but not large enough to pull out specific results.

Total travel time on a freeway facility is sensitive to a number of factors including the prevailing free-flow speed, demand levels, segment capacity, percent drop in queue discharge flow rate, demand-to-capacity ratio, weather conditions, incidents, presence of work zones, and special events. Consequently, these factors can influence travel time reliability on a freeway facility.

Exhibit 11-14 shows four cumulative TTI distributions resulting from a reliability analysis for the freeway facility given in Example Problem 1 in Chapter 25. The "recurring congestion only" curve corresponds to a reliability analysis assuming no inclement weather, incident events, or scheduled work zones in the RRP. As expected, this curve yields consistently lower (i.e., better) TTI values than do the other three TTI distributions. In this case, TTI_{95} is 1.5.

The "recurring congestion + weather" curve corresponds to an analysis in which inclement weather conditions are added to the variation in demand. As expected, this addition slightly shifts the TTI distribution toward higher TTI values without appreciably changing TTI_{95}.

The "recurring congestion + incidents" curve captures variations in demand level plus the occurrence of incidents during the RRP. As expected, the inclusion of incidents increases TTI values for the entire distribution and, consequently, results in a shift toward higher TTI values in the curve. In this case, TTI_{95} increases to about 1.8, representing a 20% increase above the base recurring-congestion case.

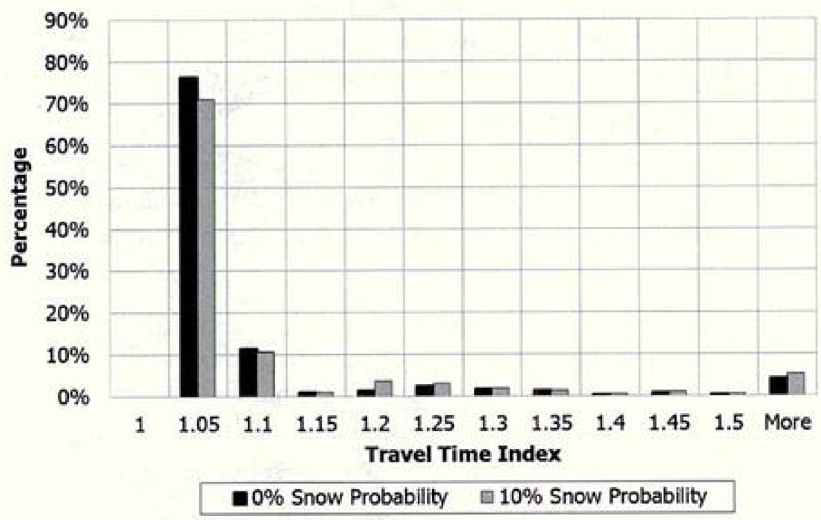
Finally, the "recurring congestion + weather + incidents" curve corresponds to an RRP that includes variations in the demand level, inclement weather events, and incidents. This curve models scenarios that combine inclement weather events, incidents, and high demand values. Therefore, the resulting TTI curve has higher TTI values than the other three curves, although again TTI_{95} does not appreciably increase compared with the "recurring congestion + incidents" case.

Exhibit 11-14
Illustrative Effects of Different Nonrecurring Sources of Congestion on the TTI Distribution

Note: Based on Example Problem 1 from Chapter 25, using default weather data for Raleigh, North Carolina, and a facilitywide incident rate of 1,050 incidents per 100 million VMT.

As shown above, the inclusion of inclement weather events in the RRP shifts the TTI distribution toward higher TTI values. Exhibit 11-15 depicts the TTI probability distribution function obtained with different weather conditions (in this case, in a city with a milder climate). Bars with a dotted pattern indicate a reliability analysis that is performed under the assumption of a 10% chance of heavy snow in December, January, and February. Dark bars correspond to an otherwise identical analysis performed under the assumption of zero snow probability in those 3 months. The exhibit shows that the higher heavy snow probability yielded a lower percentage of TTI values in the 1 to 1.05 range. A lower snow probability resulted in a lower percentage of higher TTI values.

Exhibit 11-15
Illustrative Effects of Inclement Weather Events on the TTI Distribution



Note: Based on Example Problem 1 from Chapter 25, with a facilitywide incident rate of 1,050 incidents per 100 million VMT and heavy snow probabilities of 0% and 10%.

Exhibit 11-16 illustrates the effects of incident frequency on travel time reliability. The dotted curve corresponds to the travel time distribution assuming 350 incidents per 100 million VMT. Increasing the rate from 350 to 700 incidents per 100 million VMT (dashed line) results in a shift in the TTI distribution toward a higher value. This is expected, since a greater number of scenarios are affected by incidents in this case. Increasing the rate from 700 to 1,050 incidents per million VMT (solid line) yields a further rightward shift in the distribution, as expected.

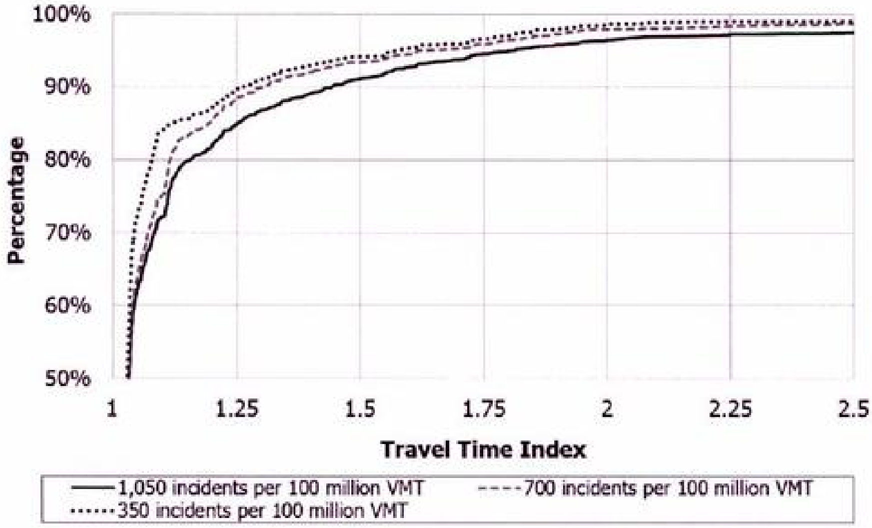


Exhibit 11-16
Illustrative Effects of Incident Rates on the TTI Distribution

Note: Based on Example Problem 1 of Chapter 25, using default weather data for Raleigh, North Carolina, and facilitywide incident rates of 350, 700, and 1,050 incidents per 100 million VMT.

The final example depicts the impacts of an ATDM strategy on travel time reliability. Exhibit 11-17 shows two TTI distributions. The first distribution is a base case (Example Problem 1 in Chapter 25), while the second is from a case where a hard shoulder running strategy is applied to the facility. As shown in the exhibit, allowing vehicles to use the shoulder shifts the TTI distribution toward lower TTI values. This trend occurs because hard shoulder running increases the capacity of the freeway facility and, as a result, travel time is consistently reduced.

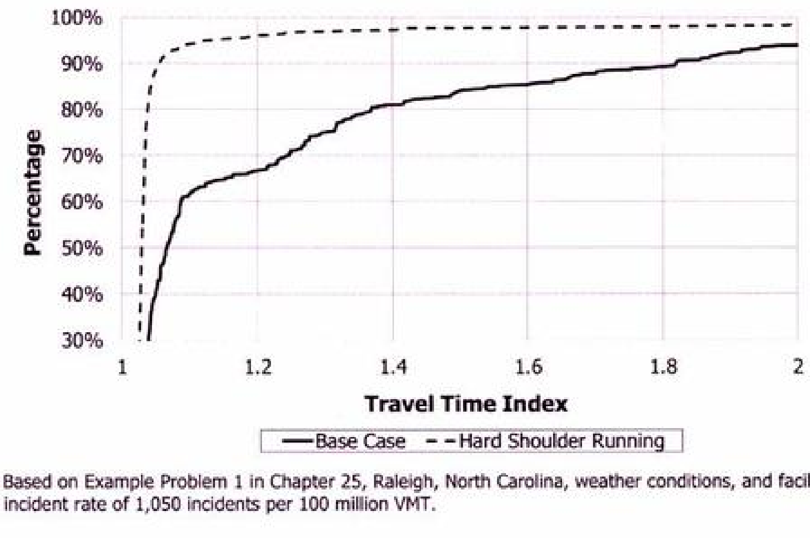


Exhibit 11-17
Effect of Activating Hard Shoulder Running ATDM Strategy

Note: Based on Example Problem 1 in Chapter 25, Raleigh, North Carolina, weather conditions, and facilitywide incident rate of 1,050 incidents per 100 million VMT.

DEFAULT VALUES

This section provides default values for much of the input data used by this chapter's reliability methodologies. Agencies are encouraged, when possible, to develop local default values on the basis of field measurements of facilities in their jurisdiction. Local defaults provide a better means of ensuring accuracy in analysis results. Facility-specific values provide the best means of ensuring an adequate representation of local and regional conditions. In the absence of local data, this section's default values can be used when the analyst believes that the values are reasonable for the facility to which they are applied.

Traffic Demand Variability

Exhibit 11-18 and Exhibit 11-19 present default demand ratios by day of week and month of year for urban and rural freeway facilities, respectively. The ratios were derived from a national freeway dataset developed by Strategic Highway Research Program 2 Project L03 (2). All ratios reflect demand relative to a Monday in January. Where possible, analysts should obtain local or regional estimates of demand variability to account for facility-specific and seasonal trends on the subject facility.

Exhibit 11-18
Default Urban Freeway Demand Ratios (ADT/Mondays in January)

	Day of Week						
Month	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**	**Saturday**	**Sunday**
January	1.00	1.00	1.02	1.05	1.17	1.01	0.89
February	1.03	1.03	1.05	1.08	1.21	1.04	0.92
March	1.12	1.12	1.14	1.18	1.31	1.13	0.99
April	1.19	1.19	1.21	1.25	1.39	1.20	1.05
May	1.18	1.18	1.21	1.24	1.39	1.20	1.05
June	1.24	1.24	1.27	1.31	1.46	1.26	1.10
July	1.38	1.38	1.41	1.45	1.62	1.39	1.22
August	1.26	1.26	1.28	1.32	1.47	1.27	1.12
September	1.29	1.29	1.32	1.36	1.52	1.31	1.15
October	1.21	1.21	1.24	1.27	1.42	1.22	1.07
November	1.21	1.21	1.24	1.27	1.42	1.22	1.07
December	1.19	1.19	1.21	1.25	1.40	1.20	1.06

Source: Derived from data presented by Cambridge Systematics et al. (*2*).
Note: Ratios represent demand relative to a Monday in January.

Exhibit 11-19
Default Rural Freeway Demand Ratios (ADT/Mondays in January)

	Day of Week						
Month	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**	**Saturday**	**Sunday**
January	1.00	0.96	0.98	1.03	1.22	1.11	1.06
February	1.11	1.06	1.09	1.14	1.35	1.23	1.18
March	1.24	1.19	1.21	1.28	1.51	1.37	1.32
April	1.33	1.27	1.30	1.37	1.62	1.47	1.41
May	1.46	1.39	1.42	1.50	1.78	1.61	1.55
June	1.48	1.42	1.45	1.53	1.81	1.63	1.57
July	1.66	1.59	1.63	1.72	2.03	1.84	1.77
August	1.52	1.46	1.49	1.57	1.86	1.68	1.62
September	1.46	1.39	1.42	1.50	1.78	1.61	1.55
October	1.33	1.28	1.31	1.38	1.63	1.47	1.42
November	1.30	1.25	1.28	1.35	1.59	1.44	1.39
December	1.17	1.12	1.14	1.20	1.43	1.29	1.24

Source: Derived from data presented by Cambridge Systematics et al. (*2*).
Note: Ratios represent demand relative to a Monday in January.

Weather Event Probabilities

Weather event probabilities by month of each weather event for the largest U.S. metropolitan areas are provided as resource material in the Technical Reference Library in online HCM Volume 4. Average durations of each severe weather event type are also provided for these metropolitan areas.

Weather Capacity and Speed Adjustment Factors

Exhibit 11-20 and Exhibit 11-21 provide default CAFs and SAFs, respectively, by weather type and facility free-flow speed. Note that the changes in CAFs and SAFs for decreasing visibility shown in the exhibit may be counterintuitive, since they are based on a single site.

The SAF is applied to the base free-flow speed, and the CAF is applied to the base capacity, both of which are calculated in the respective methodological chapters for the various freeway segment types. Both may also have been adjusted in the process of calibrating the core facility in Chapter 10. The adjustment factors below should be applied in addition to any prior CAF and SAF calibration.

Exhibit 11-20
Default CAFs by Weather Condition

		Capacity Adjustment Factors				
Weather Type	**Weather Event Definition**	**55 mi/h**	**60 mi/h**	**65 mi/h**	**70 mi/h**	**75 mi/h**
Medium rain	>0.10–0.25 in./h	0.94	0.93	0.92	0.91	0.90
Heavy rain	>0.25 in./h	0.89	0.88	0.86	0.84	0.82
Light snow	>0.00–0.05 in./h	0.97	0.96	0.96	0.95	0.95
Light–medium snow	>0.05–0.10 in./h	0.95	0.94	0.92	0.90	0.88
Medium–heavy snow	>0.10–0.50 in./h	0.93	0.91	0.90	0.88	0.87
Heavy snow	>0.50 in./h	0.80	0.78	0.76	0.74	0.72
Severe cold	<–4°F	0.93	0.92	0.92	0.91	0.90
Low visibility	0.50–0.99 mi	0.90	0.90	0.90	0.90	0.90
Very low visibility	0.25–0.49 mi	0.88	0.88	0.88	0.88	0.88
Minimal visibility	<0.25 mi	0.90	0.90	0.90	0.90	0.90
Non–severe weather	All conditions not listed above	1.00	1.00	1.00	1.00	1.00

Source: Zegeer et al. (*1*).
Note: Speeds given in column heads are free-flow speeds.

Exhibit 11-21
Default SAFs by Weather Condition

		Speed Adjustment Factors				
Weather Type	**Weather Event Definition**	**55 mi/h**	**60 mi/h**	**65 mi/h**	**70 mi/h**	**75 mi/h**
Medium rain	>0.10–0.25 in./h	0.96	0.95	0.94	0.93	0.93
Heavy rain	>0.25 in./h	0.94	0.93	0.93	0.92	0.91
Light snow	>0.00–0.05 in./h	0.94	0.92	0.89	0.87	0.84
Light–medium snow	>0.05–0.10 in./h	0.92	0.90	0.88	0.86	0.83
Medium–heavy snow	>0.10–0.50 in./h	0.90	0.88	0.86	0.84	0.82
Heavy snow	>0.50 in./h	0.88	0.86	0.85	0.83	0.81
Severe cold	<–4°F	0.95	0.95	0.94	0.93	0.92
Low visibility	0.50–0.99 mi	0.96	0.95	0.94	0.94	0.93
Very low visibility	0.25–0.49 mi	0.95	0.94	0.93	0.92	0.91
Minimal visibility	<0.25 mi	0.95	0.94	0.93	0.92	0.91
Non–severe weather	All conditions not listed above	1.00	1.00	1.00	1.00	1.00

Source: Zegeer et al. (*1*).
Note: Speeds given in column heads are free-flow speeds.

Incident Probabilities and Durations

Exhibit 11-22 provides mean distributions of freeway incidents by severity and default incident duration parameters by incident type.

Exhibit 11-22
Default Freeway Incident Severity Distribution and Duration Parameters (min)

	Incident Severity Type				
Parameter	Shoulder Closed	1 Lane Closed	2 Lanes Closed	3 Lanes Closed	4+ Lanes Closed
Distribution (%)	75.4	19.6	3.1	1.9	0
Duration (mean)	34	34.6	53.6	67.9	67.9
Duration (std. dev.)	15.1	13.8	13.9	21.9	21.9
Duration (min.)	8.7	16	30.5	36	36
Duration (max.)	58	58.2	66.9	93.3	93.3

Source: Zegeer et al. (*1*).
Notes: std. dev. = standard deviation; min. = minimum; max. = maximum.

Incident Capacity Adjustment Factors

Exhibit 11-23 shows the default CAFs associated with each incident severity. The values shown in the exhibit reflect the *remaining relative capacity per open lane*. For example, a two-lane closure incident on a six-lane directional facility (underscored) results in a loss of two full-lane capacities, in addition to maintaining only 75% of the remaining four open lanes' capacities. The result is that only three lanes worth (50%) of the facility's original six-lane capacity is maintained. No information is available on the effect of incidents on free-flow speed, so this effect is not accounted for at this time.

Exhibit 11-23
CAFs by Incident Type and Number of Directional Lanes on the Facility

Directional Lanes	No Incident	Shoulder Closed	1 Lane Closed	2 Lanes Closed	3 Lanes Closed	4 Lanes Closed
2	1.00	0.81	0.70	N/A	N/A	N/A
3	1.00	0.83	0.74	0.51	N/A	N/A
4	1.00	0.85	0.77	0.50	0.52	N/A
5	1.00	0.87	0.81	0.67	0.50	0.50
6	1.00	0.89	0.85	<u>0.75</u>	0.52	0.52
7	1.00	0.91	0.88	0.80	0.63	0.63
8	1.00	0.93	0.89	0.84	0.66	0.66

Source: Zegeer et al. (*1*).
Notes: N/A = not applicable — the number of lanes closed equals or exceeds the number of directional lanes. The methodology does not permit all directional lanes of a facility to be closed.

PLANNING, PRELIMINARY ENGINEERING, AND DESIGN ANALYSIS

A facility's average travel time will vary from hour to hour, day to day, and season to season, depending on fluctuations in demand, weather, incidents, and work zones. Reliability measures characterize this distribution of travel times for a selected period of a year meaningful to the analyst, the agency's objectives, and the general public.

Estimating performance measures requiring complex calculations, such as the reliability distribution described in this chapter, can be challenging in a planning context. However, two options exist for applying this chapter's reliability methodology in a planning context:

1. Application of HCM methods using default values and
2. Simplified percentile estimation method.

Both methods are introduced below and are described further in the *Planning and Preliminary Engineering Applications Guide to the HCM*, available in the Technical Reference Library in the online HCM Volume 4.

HCM Method Using Default Values

This chapter's method for estimating travel time reliability can, to some extent, be automated through the use of default values. Automating the generation of inputs, along with applying the method in a computational engine or software, allows reliability performance to be estimated with minimal input needs, which may make the process suitable for application in a planning context. Exhibit 11-24 lists the required input data and describes where default values are provided.

Exhibit 11-24
Input Data Needs for HCM Planning Reliability Analysis of Freeways

Data Category	Description	Data Source
Time periods	Analysis period, study period, reliability reporting period	Must be selected by the analyst
Demand patterns	Day-of-week by month-of-year demand factors	Default values provided in Chapter 25
Weather	Probabilities of various intensities of rain, snow, cold, and low visibility by month	Data sources and default values provided in Chapter 25
Incidents	Crash rate and incident-to-crash ratio for the facility, in combination with defaulted incident type probability and duration data	Crash rate must be provided; default values available in Chapter 25 for other data
Work zones and special events	Changes to base conditions and schedule	Must be specified when relevant to the analysis
Nearest city	City with airport weather station	Required to apply weather defaults
Traffic counts	Demand multiplier for demand represented in base dataset	Must be provided

Simplified Method

The equations in this section can be used to estimate specific TTI percentiles as an approximation of freeway facility reliability (*11, 12*). This method does not specify the full reliability distribution, nor is it customized to a specific facility's geometry or operating characteristics.

First, the mean annual travel time index, including incident effects, is computed:

$$TTI_{\text{mean}} = 1 + FFS \times (RDR + IDR)$$

Equation 11-1

where

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

FFS = free-flow speed (mi/h);

RDR = recurring delay rate (h/mi), from Equation 11-2; and

IDR = incident delay rate (h/mi), from Equation 11-3.

$$RDR = \frac{1}{S} - \frac{1}{FFS}$$

Equation 11-2

$$IDR = [0.020 - (N - 2) \times 0.003] \times X^{12}$$

Equation 11-3

where

S = peak-hour speed (mi/h),

N = number of lanes in one direction ($N = 2$ to 4), and

X = peak hour volume-to-capacity ratio (decimal).

Equation 11-3 is valid only for $X \leq 1.00$ and $N = 2$, 3, or 4. Values of X greater than 1.00 should be capped at 1.00, and values of N greater than 4 should be capped at 4, for use in Equation 11-3.

The 95th percentile travel time index (TTI_{95}) and percent of trips traveling under 45 mi/h (PT_{45}) can be computed from the average annual TTI according to the following equations.

Equation 11-4

$$TTI_{95} = 1 + 3.67 \times \ln\left(TTI_{\text{mean}}\right)$$

Equation 11-5

$$PT_{45} = 1 - \exp\left[-1.5115 \times \left(TTI_{\text{mean}} - 1\right)\right]$$

where

TTI_{95} = 95th percentile TTI (unitless),

TTI_{mean} = average annual mean travel time index (unitless), and

PT_{45} = percent of trips that occur at speeds less than 45 mi/h (decimal).

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and level-of-service analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for applying alternative tools to the analysis of freeway facilities. Additional information on this topic may be found in Chapter 25, Freeway Facilities: Supplemental.

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

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

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

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

- Scenarios may be defined differently from and may be of longer or shorter duration than those used in HCM analysis.
- Incident start times and durations will no longer need to be rounded to the nearest 15-min analysis period.
- Weather start times and durations will no longer need to be rounded to the nearest 15-min analysis period.
- Demand will no longer need to be held constant for the duration of the 15-min analysis period.
- The peak hour factors used to identify the peak 15-min flow rate within the hour will no longer be applied. They will be replaced with the analysis tool's built-in randomization process.
- This chapter's recommended freeway capacity adjustment factors, along with the free-flow speed adjustment factors for weather events and incidents, will have to be converted by the analyst to the microsimulation model equivalents: desired speed distribution and desired headway distribution. Acceleration and deceleration rates will also be affected for some weather events.
- This chapter's recommended freeway speed–flow curves for weather events and incidents will be replaced with adjustments to the model's car-following parameters, such as desired free-flow speed, saturation headway, and start-up lost time. Unlike incidents, which the tool's car-following logic can address, weather is modeled by adjusting the car-following parameters through weather adjustment factors before the scenarios are run. Application guidance and typical factors are provided in FHWA's *Traffic Analysis Toolbox* (*13*).

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

6. REFERENCES

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

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12. Cambridge Systematics, Inc. *IDAS User's Manual,* Table B.2.14. http://idas.camsys.com/documentation.htm. Accessed Aug. 14, 2014.

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CHAPTER 12
BASIC FREEWAY AND MULTILANE HIGHWAY SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

This chapter presents methodologies for analyzing the capacity and level of service (LOS) of basic freeway and multilane highway segments. These segments are outside the influence of merging, diverging, and weaving maneuvers. In the case of multilane highways, they are also outside the influence of signalized intersections. Because of the similar operational characteristics of basic freeway and multilane highway segments, they are analyzed with the same methodology. The similarities include a common form of the speed–flow relationship and the effects attributed to the number of lanes, lane width, lateral clearance, and the presence of heavy vehicles. The chapter also provides methods for analyzing basic managed lane segments on freeways and bicycle LOS on multilane highways.

This chapter focuses on *uninterrupted flow,* which refers to access-controlled facilities, with access and egress being controlled through grade-separated cross streets and ramp movements to access the facility. For multilane highways, uninterrupted flow also exists when there are no traffic control devices that interrupt traffic and where no platoons are formed by upstream traffic signals. Typically, this condition occurs when the multilane highway segment is 2 mi or more from the nearest traffic signal.

The methodologies in this chapter are limited to *uncongested flow* conditions. Uncongested flow conditions require that the demand-to-capacity ratio for the segment be less than or equal to 1.0. Uncongested flow on freeways and multilane highways further means that there are no queuing impacts on the segment from downstream bottlenecks. Chapter 10, Freeway Facilities Core Methodology, provides an evaluation method for analyzing oversaturated basic freeway segments. The *Highway Capacity Manual* (HCM) does not currently provide a method for evaluating oversaturated multilane highways other than to identify them as LOS F.

CHAPTER ORGANIZATION

Section 2 of this chapter presents the basic concepts of freeway and multilane uninterrupted-segment operations, including the definition of base conditions; differences in the treatment of basic freeway and multilane segments; basic managed lane concepts; speed–flow relationships; and demand, capacity, and LOS measures for automobile traffic.

Section 3 presents the base methodology for evaluating automobile operations on basic freeway and multilane highway segments.

Section 4 extends the core method presented in Section 3 to applications for managed lanes, including high-occupancy vehicle (HOV) and high-occupancy/toll (HOT) lanes (also called express or priced managed lanes) with various types of separation from the general purpose lanes. This method is based on findings from National Cooperative Highway Research Program (NCHRP) Project 03-96 (1–3). Additional extensions include the effect of trucks and other heavy vehicles on capacity and LOS and a method for evaluating bicycle LOS on multilane highways (with details provided in Chapter 15, Two-Lane Highways).

Section 5 presents application guidance on using the results of basic freeway and multilane highway segment analysis, including example results from the methods, information on the sensitivity of results to various inputs, and a service volume table for freeway and multilane highway segments.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 3, Modal Characteristics, where the motorized vehicle "Variations in Demand" subsection describes typical travel demand patterns for freeway and multilane highway segments.
- Chapter 4, Traffic Operations and Capacity Concepts, which provides background for the speed, flow, density, and capacity terms specific to freeway and multilane highway segments that are presented in this chapter's Section 2.
- Chapter 10, Freeway Facilities Core Methodology, and Chapter 11, Freeway Reliability Analysis, which use the basic freeway segment methodology described in this chapter in analyzing a larger facility comprising freeway basic, merge and diverge, weaving, and managed lane segments over extended time periods.
- Chapter 11, Freeway Reliability Analysis, which provides a method for evaluating freeway facilities with basic segments in a reliability context. The chapter also provides default speed and capacity adjustment factors that can be applied in this chapter's methodology.
- Chapter 25, Freeway Facilities: Supplemental, which presents a method for evaluating mixed truck and automobile traffic streams on composite grades.
- Chapter 26, Freeway and Highway Segments: Supplemental, which provides state-specific heavy vehicle percentages, presents a method for evaluating mixed truck and automobile traffic streams on single grades, describes capacity and speed adjustments for driver populations unfamiliar with a roadway, provides guidance for measuring freeway capacity in the field, and presents example problems with step-by-step calculations using this chapter's methods.
- Case Study 4, New York State Route 7, in the *HCM Applications Guide* in Volume 4, which demonstrates how this chapter's methods can be applied to the evaluation of an actual freeway facility.
- Section H, Freeway Analyses, and Section I, Multilane Highways, of the *Planning and Preliminary Engineering Applications Guide to the HCM,* found in Volume 4, which describes how to incorporate this chapter's methods and performance measures into a planning or preliminary engineering effort.

2. CONCEPTS

BASIC FREEWAY AND MULTILANE HIGHWAY SEGMENT DESCRIPTION

A basic freeway or multilane highway segment is outside the influence area of any merge, diverge, or weaving segments and of any signalized intersections. Exhibit 12-1 shows typical basic freeway segment cross sections, and Exhibit 12-2 illustrates common types of multilane highways.

Exhibit 12-1
Basic Freeway Segment Types Illustrated

Source: © 2014 Google

(a) Eight-Lane Urban Freeway Segment

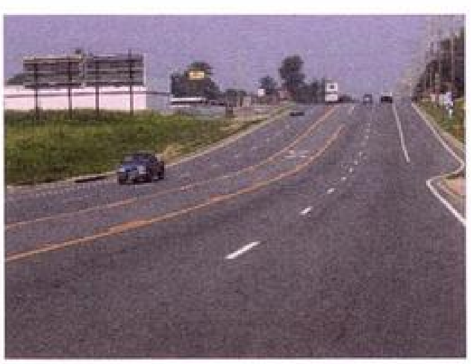
Source: © 2014 Google

(b) Six-Lane Rural Freeway Segment

Exhibit 12-2
Multilane Highway Types Illustrated

(a) Divided Suburban Multilane Highway Segment

(b) Undivided Suburban Multilane Highway Segment



(c) Suburban Multilane Highway Segment with Two-Way Left-Turn Lane

(d) Undivided Rural Multilane Highway Segment

Analysis segments must have uniform geometric and traffic conditions, including demand flow rates.

Basic freeway segments generally have four to eight lanes (in both directions) and posted speed limits between 50 and 75 mi/h. The median type depends on right-of-way constraints and other factors.

Multilane highways generally have four to six lanes (in both directions) and posted speed limits between 40 and 55 mi/h. In some states, speed limits of 60 or 65 mi/h or higher are used on some multilane highways. These highways may be undivided (with only a centerline separating the directions of flow) or divided (with a physical median separating the directions of flow), or they may have a two-way left-turn lane (TWLTL). Typically they are located in suburban areas and lead into city centers or in high-volume rural corridors, where they connect two cities or activity centers that generate a substantial number of daily trips.

All analyses are applied to segments with uniform characteristics. Uniform segments must have the same geometric and traffic characteristics, including a constant demand flow rate.

Influence Areas of Merge, Diverge, and Weaving Segments

In general terms, the influence area of merge (on-ramp) segments extends 1,500 ft downstream of the merge point. The influence area of diverge (off-ramp) segments extends 1,500 ft upstream of the diverge point. The influence area of weaving segments extends 500 ft upstream and downstream of the gore-to-gore segment length. For undersaturated operations, these distances define the areas most affected by merge, diverge, and weaving movements. A complete discussion of these influence areas is provided in Chapter 10, Freeway Facilities Core Methodology, with additional discussion in Chapter 13, Freeway Weaving Segments, and Chapter 14, Freeway Merge and Diverge Segments.

Influence of Breakdowns in Adjacent Freeway Segments

The impact of breakdowns in any type of freeway segment on an adjacent basic segment can be addressed with the methodologies of Chapter 10, Freeway Facilities Core Methodology, and Chapter 11, Freeway Reliability Analysis. Breakdown events are defined in more detail below.

Influence of Traffic Signals on Multilane Highway Segments

The influence area of traffic signals on multilane highways is typically about 1 mi, which means that uninterrupted flow may exist if traffic signals are spaced 2 mi or more apart. Many multilane highways will have periodic signalized intersections, even if the average signal spacing is well over 2 mi. In such cases, the multilane highway segments that are more than 2 mi away from any traffic signals are analyzed with this chapter's methodology. Isolated signalized intersections along multilane highways should be analyzed with the methodology of Chapter 19, Signalized Intersections.

FLOW CHARACTERISTICS OF BASIC FREEWAY AND MULTILANE HIGHWAY SEGMENTS

Traffic flow within basic freeway segments can be highly dependent on the conditions constricting flow at upstream and downstream bottleneck locations. Such bottlenecks can be created by any or by a combination of the following: merging, diverging, or weaving traffic; lane drops; maintenance and construction activities; traffic accidents or incidents; objects in the roadway; and geometric characteristics such as upgrades or sharp horizontal curves. Bottlenecks can exist even when a lane is not fully blocked. Partial blockages will cause drivers to slow and divert their paths. In addition, the practice of rubbernecking near roadside incidents or accidents can cause functional bottlenecks. Many nonrecurring congestion effects have a facilitywide impact and therefore are considered in Chapter 10.

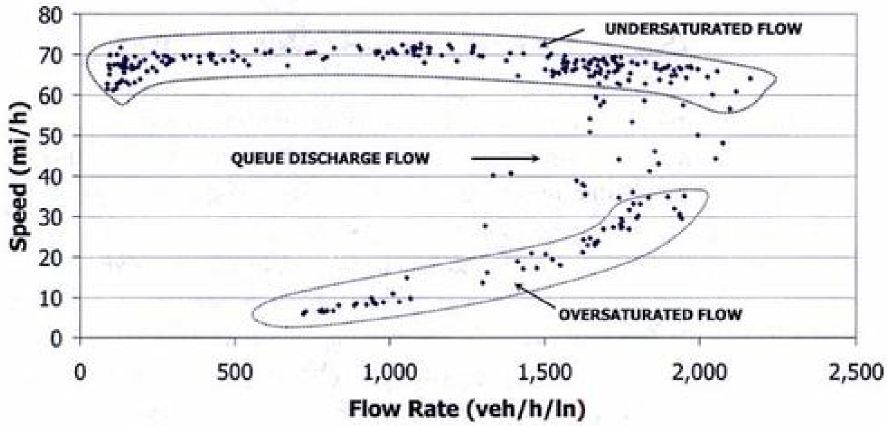
Uninterrupted flow on multilane highways is similar to that on basic freeway segments. However, there are several important differences. Because side frictions are present in varying degrees from uncontrolled driveways and intersections, as well as from opposing flows on undivided cross sections, speeds on multilane highways tend to be lower than those on similar basic freeway segments. The basic geometry of multilane highways tends to be more constrained than that of basic freeway segments, consistent with lower speed expectations. Finally, isolated signalized intersections can exist along multilane highways. The overall result is that speeds and capacities on multilane highways are lower than those on basic freeway segments with similar cross sections.

As was discussed in more detail in Chapter 4, Traffic Operations and Capacity Concepts, traffic flow within a basic freeway or multilane highway segment can be categorized as one of three general types: undersaturated, queue discharge, and oversaturated.

- *Undersaturated flow* represents conditions under which the traffic stream is unaffected by upstream or downstream bottlenecks.
- *Queue discharge flow* represents congested traffic flow that has just passed through a bottleneck and is accelerating back to the drivers' desired speeds. If no other downstream bottleneck exists, queue discharge flow will be relatively stable until the queue is fully discharged.
- *Oversaturated flow* represents the conditions within a queue that has backed up from a downstream bottleneck. These flow conditions do not reflect the prevailing conditions of the segment itself but rather the consequences of a downstream problem. All oversaturated flow is considered to be congested.

An example of each of the three types of flow discussed is illustrated in Exhibit 12-3, which uses data from a freeway segment in California.

Exhibit 12-3
Three Types of Flow on a Basic Freeway Segment



Source: California Department of Transportation, 2008.
Note: I-405, Los Angeles, California.

FREEWAY CAPACITY DEFINITIONS

Freeway segment capacity is commonly understood to be a maximum flow rate associated with the occurrence of some type of breakdown, which results in lower speeds and higher densities. Previous research has shown that when oversaturation begins, queues develop and vehicles discharge from the bottleneck at a queue discharge rate that is usually lower than the throughput rate before the breakdown. This is also known as the "capacity drop phenomenon." Several key terms related to freeway capacity are defined below as they apply to this chapter. Details on the measurement of breakdown and capacities are provided in Section 5 of Chapter 26, Freeway and Highway Segments: Supplemental.

Freeway Breakdown

A freeway flow breakdown describes the transition from uncongested to congested conditions. The formation of queues upstream of the bottleneck and the reduced prevailing speeds make the breakdown evident.

In the HCM freeway methodology, a breakdown event on a freeway bottleneck is defined as a sudden drop in speed of at least 25% below the free-flow speed (FFS) for a sustained period of at least 15 min that results in queuing upstream of the bottleneck.

Recovery

A freeway segment is considered to have recovered from the breakdown event and the resulting oversaturated conditions when the average speeds (or occupancies) reach prebreakdown conditions for a minimum duration of 15 min. The definition of recovery is therefore the inverse of the definition of breakdown, requiring a recovery to near prebreakdown conditions (operations above the speed threshold) for at least 15 min.

The HCM defines the breakdown recovery on a freeway bottleneck as a return of the prevailing speed to within 10% of the FFS for a sustained period of at least 15 min, without the presence of queuing upstream of the bottleneck.

Prebreakdown Flow Rate

The prebreakdown flow rate is the flow rate that immediately precedes the occurrence of a breakdown event. The literature suggests that this flow rate does not have a fixed value, since evidence shows that breakdowns are stochastic in nature and could occur following a range of flow rates. The flow rate is typically expressed in units of passenger cars per hour per lane (pc/h/ln) by converting trucks and other heavy vehicles into an equivalent passenger car traffic stream.

In the HCM, the prebreakdown flow rate is defined as the 15-min average flow rate immediately before the breakdown event. For the purpose of this chapter, the prebreakdown flow rate is equivalent to the segment capacity.

Postbreakdown Flow Rate or Queue Discharge

The postbreakdown flow rate is also referred to as the *queue discharge flow rate* or the average discharge flow rate. This flow rate is usually lower than the prebreakdown flow rate, resulting in significant loss of freeway throughput during congestion. Cases where the postbreakdown flow rate exceeds the prebreakdown flow rate have also been observed, mostly when the prebreakdown flow rate is low. Studies have indicated that the average difference between the postbreakdown and the prebreakdown flow rates varies from as little as 2% to as much as 20%, with a default value of 7% recommended.

In the HCM the queue discharge rate is defined as the average flow rate during oversaturated conditions (i.e., during the time interval after breakdown and before recovery).

CAPACITY UNDER BASE CONDITIONS

The base conditions under which the full capacity of a basic freeway or multilane highway segment is achieved include good weather, good visibility, no incidents or accidents, no work zone activity, and no pavement deterioration serious enough to affect operations. The term "base conditions" presupposes the existence of these conditions. If any of these conditions does not exist, the speed and capacity of the freeway segment can be adjusted through this chapter's methodology to reflect prevailing conditions. Base conditions also include the following:

- No heavy vehicles in the traffic stream,
- A driver population mostly composed of regular users who are familiar with the facility, and
- 12-ft lane widths and adequate lateral clearances (different for freeway and multilane highways).

The capacity of a basic freeway segment under base conditions varies with the FFS. Exhibit 12-4 gives capacity values under base conditions for a selection of FFS values. Interpolation between FFS values is permitted. In all cases, capacity represents a maximum flow rate for a 15-min interval.

Exhibit 12-4
Basic Freeway and Multilane Highway Segment Capacity Under Base Conditions

FFS (mi/h)	Capacity of Basic Freeway Segments (pc/h/ln)	Capacity of Multilane Highway Segments (pc/h/ln)
75	2,400	NA
70	2,400	2,300[a]
65	2,350	2,300[a]
60	2,300	2,200
55	2,250	2,100
50	NA	2,000
45	NA	1,900

Notes: NA = not available.
[a] Capacities for multilane highways with 65- and 70-mi/h FFS are extrapolated and not based on field data.

It is reiterated that *these base capacities reflect ideal conditions on a facility without any capacity-reducing effects*. For example, the base capacities assume no heavy vehicles; no grades; and no additional friction effects due to poor pavement conditions, narrow lanes, or lighting conditions. Furthermore, the capacities shown in Exhibit 12-4 apply to a peak 15-min period (expressed as hourly flow rates); capacities measured over a 1-h period may be less than these values. Finally, the base capacities do not include the effects of nonrecurring sources of congestion, such as severe weather, incidents, or work zones. Therefore, calibration of the base capacity to reflect local conditions is important, especially when a segment is evaluated in the context of an extended freeway facility. For some adjustments, the HCM method provides explicit guidance. In other cases, available defaults for adjustment factors are limited, and these values should therefore be obtained by using local data.

Base capacity values refer to the average flow rate across all lanes without impacts of heavy vehicles, grades, or other sources of friction.

Since freeways usually do not operate under base conditions, observed capacity values will typically be lower than the base capacity values. Local calibration of capacity values is critical to ensure proper evaluation of basic freeway segments, especially in the context of an extended freeway facility.

Chapters 10 and 11 provide additional information allowing capacity values to be adjusted to reflect the impact of long- and short-term construction and maintenance activities, adverse weather conditions, accidents or incidents, and the use of active traffic and demand management.

Capacity varies stochastically, and any given location could have a larger or smaller value than the base capacity.

Capacities represent an average flow rate across all lanes. Individual lanes could have higher stable flows.

The base capacity values represent national norms. Capacity varies stochastically, and any given location could have a larger or smaller value. Furthermore, capacity refers to the *average flow rate across all lanes*. Thus, a three-lane basic freeway segment with a 70-mi/h FFS would have an expected base capacity of 3 × 2,400 = 7,200 pc/h. This flow would not be uniformly distributed across all lanes. Thus, one or two lanes could have stable base flows in excess of 2,400 pc/h/ln. Similarly, a two-lane (in one direction) multilane highway segment with a 60-mi/h FFS would have an expected capacity of 2 × 2,200 = 4,400 pc/h. This flow would not be uniformly distributed. Thus, one lane could have stable flows in excess of 2,200 pc/h/ln.

Density at capacity for both basic freeway and multilane highway segments occurs at about 45 pc/mi/ln, or at an average vehicle spacing of 117 ft.

Basic freeway and multilane highway segments reach their capacity at a density of approximately 45 pc/mi/ln, although this value varies somewhat from location to location. At this density, vehicles are spaced too closely to dampen the impact of any perturbation in flow, such as a lane change or a vehicle entering the roadway, without causing a disruption in flow that propagates upstream.

In a freeway facility context (Chapter 10), a basic freeway segment typically does not break down unless a work zone, incident, or geometric constraint results in a reduction of the segment's capacity relative to adjacent segments. More commonly, the throughput of the basic freeway segment is dictated by upstream or downstream merge, diverge, or weaving segments that tend to govern the operations (and capacity) of the facility.

SPEED–FLOW RELATIONSHIP

Characteristics such as lane width, lateral clearance, median type, and (in the case of multilane highways) access point density will affect the FFS of the facility. Changes in the FFS further translate into different speed–flow curves describing operations under base conditions at higher volume levels.

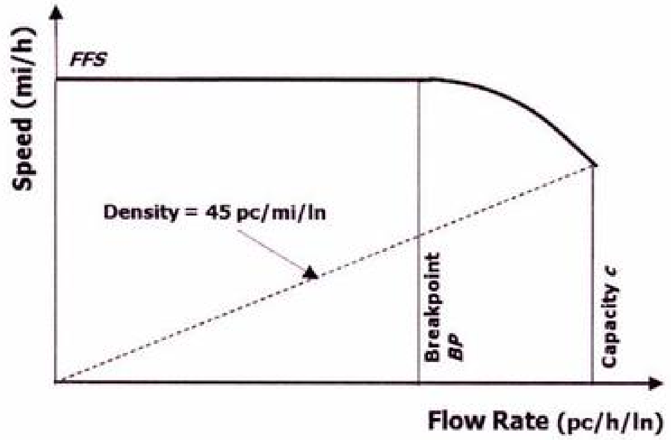
The methodology provides adjustments for situations when the base conditions do not apply.

Under base conditions, speed–flow curves for uninterrupted flow on basic freeway and multilane highway segments follow a common form:

- *Constant speed range.* There is a range of flow rates (in passenger cars per hour per lane) over which speed is constant. The range extends from a flow rate of zero to a breakpoint value *BP*. Over this range, the speed is equal to the FFS.
- *Decreasing speed range.* From *BP* to the capacity *c*, speed decreases from the FFS in a generally parabolic relationship.
- *Capacity.* In all cases, capacity occurs when the traffic stream density *D* is 45 pc/mi/ln, indicated by the dashed line in Exhibit 12-5.

The general form of this relationship is illustrated in Exhibit 12-5, where the *x*-axis represents the adjusted 15-min demand flow rate v_p (pc/h/ln) and the *y*-axis represents the space mean speed *S* of the traffic stream (mi/h). The equation for the base speed–flow curve for every basic freeway and multilane highway segment follows this form. In all cases, the value of capacity is directly related to the FFS. For basic freeway segments, the value of *BP* is also directly related to the FFS. For multilane highway segments, the breakpoint value is a constant value, occurring at 1,400 pc/h/ln.

Exhibit 12-5
General Form for Speed–Flow Curves on Basic Freeway and Multilane Highway Segments



The general analytic form of the speed–flow relationship is given by Equation 12-1, while the equations for determining the model parameters, including the breakpoint and the capacity—both of which are based on FFS—are given in Exhibit 12-6. The capacity adjustment (CAF) and speed adjustment factors (SAF) shown in Exhibit 12-6 are calibration parameters used to adjust for local conditions or to account for nonrecurring sources of congestion, and they are discussed in the core methodology section of this chapter. The CAF and SAF adjustments are only provided for basic freeway segments, since no empirical research exists for equivalent capacity-reducing effects on multilane highways.

Equation 12-1

$$S = FFS_{adj} \qquad v_p \leq BP$$

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{c_{adj}}{D_c}\right)(v_p - BP)^a}{(c_{adj} - BP)^a} \qquad BP < v_p \leq c$$

where S is the mean speed of the traffic stream under base conditions (mi/h) and other variables are as given in Exhibit 12-6.

The development and calibration of speed–flow curves for basic freeway and multilane highway segments and the development of a common form for representing these curves are described elsewhere (4–7). Basic speed–flow curves have been developed for FFS values between 55 and 75 mi/h for freeways and for FFS values between 45 and 70 mi/h for multilane highways (however, the 65- and 70-mi/h curves should be used with caution since data for those conditions are limited).

Exhibit 12-6
Parameters for Speed–Flow Curves for Basic Freeway and Multilane Highway Segments

Parameter	Definition and Units	Basic Freeway Segments	Multilane Highway Segments
FFS	Base segment free-flow speed (mi/h)	Measured OR predicted with Equation 12-2	Measured OR predicted with Equation 12-3
FFS_{adj}	Adjusted free-flow speed (mi/h)	$FFS_{adj} = FFS \times SAF$	No adjustments
SAF	Speed adjustment factor (decimal)	Locally calibrated OR estimated with Chapter 11; $SAF = 1.00$ for base conditions	1.00
c	Base segment capacity (pc/h/ln)	$c = 2{,}200 + 10(FFS - 50)$ $c \leq 2{,}400$ $55 \leq FFS \leq 75$	$c = 1{,}900 + 20(FFS - 45)$ $c \leq 2{,}300$ $45 \leq FFS \leq 70$
c_{adj}	Adjusted segment capacity (pc/h/ln)	$c_{adj} = c \times CAF$	No adjustments
CAF	Capacity adjustment factor (decimal)	Locally calibrated OR estimated with Chapter 11; $CAF = 1.00$ for base conditions	1.00
D_c	Density at capacity (pc/mi/ln)	45	45
BP	Breakpoint (pc/h/ln)	$BP_{adj} = [1{,}000 + 40 \times (75 - FFS_{adj})] \times CAF^2$	1,400
a	Exponent calibration parameter (decimal)	2.00	1.31

The largest difference in the speed–flow curves for basic freeway and multilane highway segments is in the breakpoint. For freeways, the breakpoint varies with FFS—specifically, the breakpoint *increases* as the FFS *decreases*. This suggests that at lower values of FFS, drivers will maintain the FFS through higher flow levels. For multilane highways, the breakpoint is a constant. Exhibit 12-7 and Exhibit 12-8 show the base speed–flow curves for basic freeway and multilane highway segments, respectively, for 5-mi/h increments of FFS.

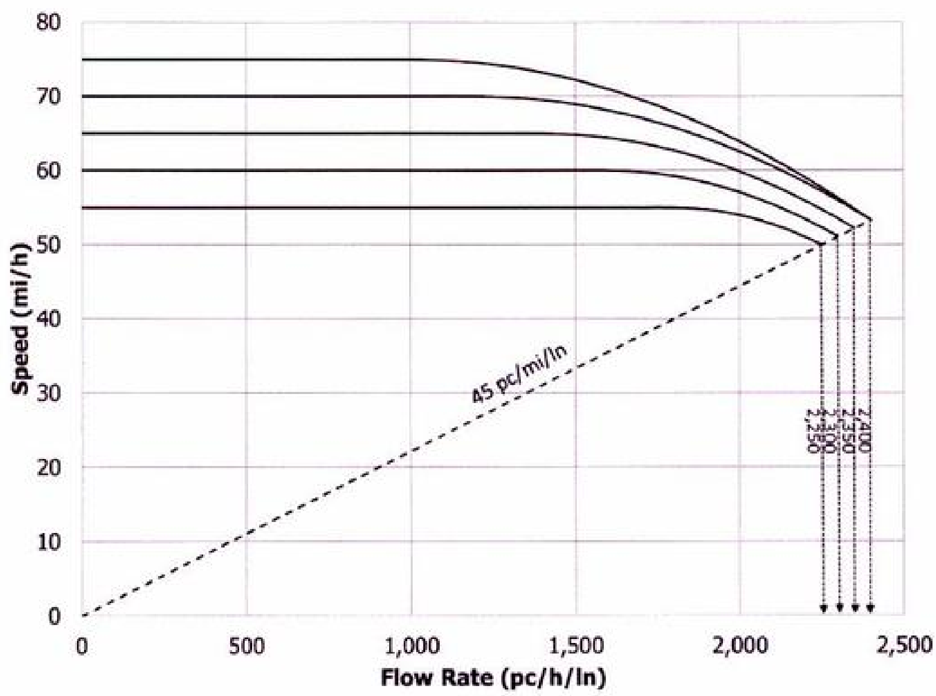


Exhibit 12-7
Speed–Flow Curves for Basic Freeway Segments

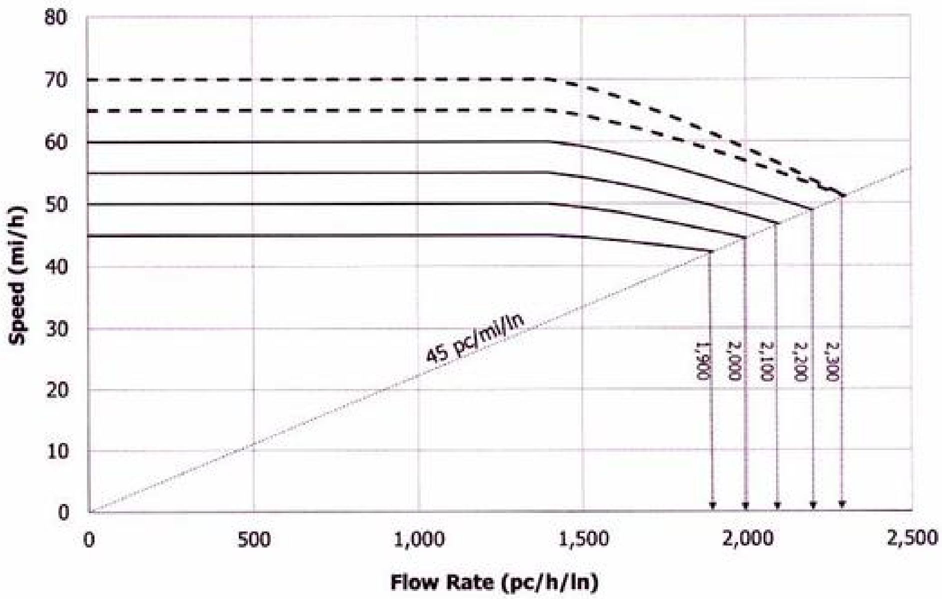


Exhibit 12-8
Speed–Flow Curves for Multilane Highway Segments

Note: Dashed curves are extrapolated and not based on field data.

BASIC MANAGED LANE SEGMENT CONCEPTS

Types of Managed Lane Segments

Managed lane segments may include HOV lanes, HOT lanes, or express toll lanes. The vehicle composition, driver type, FFS, capacity, and driver behavior characteristics of managed lane traffic streams are different from those of general purpose lanes. In addition, interaction occurs between the two traffic streams, especially when there is no physical barrier between the managed and the general purpose lanes (1–3).

Five types of basic managed lane segments are identified, on the basis of the number of managed lanes and the type of separation from the general purpose lanes. The speed–flow characteristics of each basic managed lane segment type are different. The five segment types are illustrated in Exhibit 12-9 and consist of the following:

1. *Continuous access:* Skip-stripe or solid single line–separated, single lane;
2. *Buffer 1:* Buffer-separated, single lane;
3. *Buffer 2:* Buffer-separated, multiple lanes;
4. *Barrier 1:* Barrier-separated, single lane; and
5. *Barrier 2:* Barrier-separated, multiple lanes.

Basic Managed Lane Segment Capacity

The capacity of managed lanes can be difficult to ascertain because they are often designed to operate at high levels of service and below capacity. While managed lanes do fail, empirical data on their true capacity values are limited. HOT lane users are provided with an incentive to pay for the use of the lane in return for achieving reliable travel times. Research (1–3) has documented the maximum observed 15-min hourly flow rates (without any breakdowns observed) on basic managed lane segments, and these values are documented in this chapter as the "capacity." Actual managed lane segment capacity, therefore, may be underestimated in some cases. Users of the HCM are encouraged to calibrate parameters to reflect local conditions. In this chapter's methodologies, the speed–flow curves for both managed and general purpose lanes can be modified to account for local measurements of capacity, FFS, or both.

The capacity of a basic managed lane segment depends on the number of lanes on the segment. A single-lane managed lane segment does not offer the opportunity to pass slower vehicles, which greatly reduces its capacity and affects its speed–flow relationship. Capacity is also highly dependent on the type of separation between the managed and general purpose lanes, with barrier-separated managed lanes less susceptible to operational conditions in the general purpose lanes than other types of managed lanes (continuous access, marking-only, and buffer-separated). This effect is discussed in more detail below.

Exhibit 12-9
Basic Managed Lane Segment Types

Continuous Access

Source: ©2014 Google.
Note: I-5, Seattle, Washington.

Buffer 1

Source: ©2014 Google.
Note: I-394, Minneapolis, Minnesota.

Buffer 2

Source: ©2014 Google.
Note: I-110, Los Angeles, California.

Barrier 1

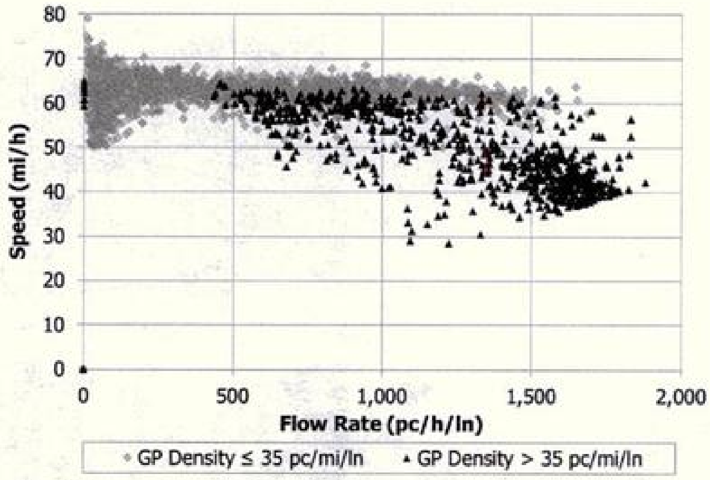
Source: ©2014 Google.
Note: I-5, Orange County, California.

Barrier 2

Source: ©2014 Google.
Note: I-5, Seattle, Washington.

Exhibit 12-10 shows how the speed–flow relationship at high flows diverges for a continuous access basic managed lane segment once the neighboring general purpose lanes approach capacity. Divergence typically occurs when the general purpose lane density exceeds 35 pc/mi/ln, which is the threshold for entering LOS E. This interaction starts even at low flow rates on the managed lane at about 500 pc/h/ln. Managed lanes with barrier separation, on the other hand, operate virtually the same as general purpose lanes and do not appear to be sensitive to high densities in the general purpose lanes.

Exhibit 12-10
Continuous Access Managed Lane Speed–Flow Data With and Without the General Purpose Lane Approaching Capacity



Note: GP = general purpose lane.

Exhibit 12-11 provides estimated capacities for basic managed lane segments as a function of the FFS and separation from the general purpose lanes. As mentioned above, these values represent the maximum observed flow rates from a national study of managed lane segments (1–3) but are not necessarily associated with a density of 45 pc/h/ln.

Exhibit 12-11
Estimated Lane Capacities for Basic Managed Lane Segments

FFS	Estimated Lane Capacities (pc/h/ln) by Basic Managed Lane Segment Type				
(mi/h)	Continuous Access	Buffer 1	Buffer 2	Barrier 1	Barrier 2
75	1,800	1,700	1,850	1,750	2,100
70	1,750	1,650	1,800	1,700	2,050
65	1,700	1,600	1,750	1,650	2,000
60	1,650	1,550	1,700	1,600	1,950
55	1,600	1,500	1,650	1,550	1,900

An example illustration of the resulting speed–flow curves for a managed lane segment with continuous access is shown in Exhibit 12-12. An illustration and comparison of the speed–flow relationships for different types of managed lanes are shown in Exhibit 12-13. The parameters used to obtain these curves are presented later in Exhibit 12-30.

In both exhibits, the *frictional effect* refers to a managed lane that is affected by elevated density in the general purpose lanes (i.e., densities greater than 35 pc/mi/ln). This frictional effect only applies to some of the managed lane types and specifically does not occur for barrier-separated managed lanes or two-lane managed lanes with buffer separation.

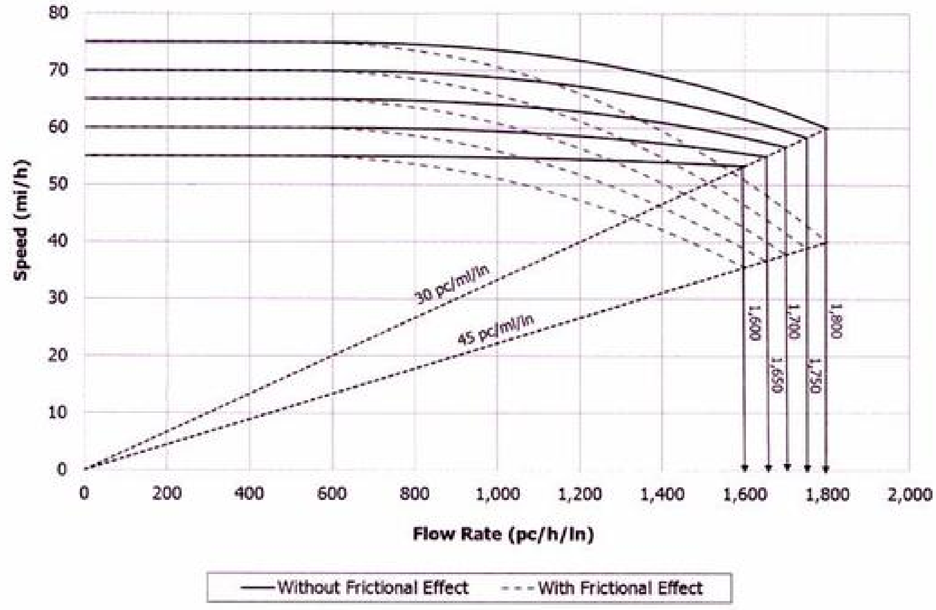


Exhibit 12-12
Example Speed–Flow Relationships for a Continuous Access Managed Lane Segment

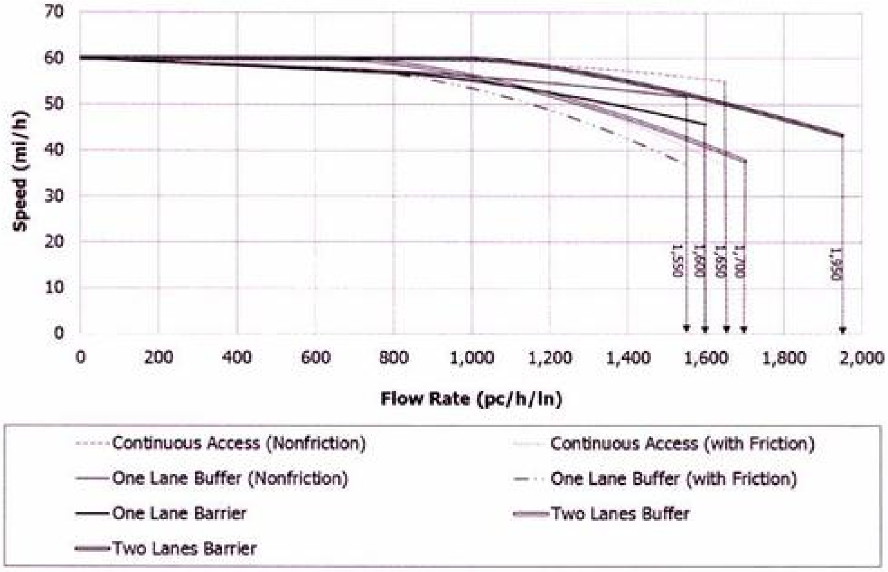


Exhibit 12-13
Speed–Flow Curve Comparison for Managed Lane Segment Types with 60-mi/h FFS

HEAVY VEHICLE CONCEPTS

The traffic performance of heavy vehicles is significantly different from that of automobiles. The differences relate to vehicle acceleration and deceleration characteristics, as reflected in their weight-to-power ratios and lengths. Two categories of heavy vehicles are defined: single-unit trucks (SUTs) and tractor-trailers (TTs). Buses and recreational vehicles are treated as SUTs in the HCM. Chapter 3, Modal Characteristics, provides a more detailed discussion of the types of heavy vehicles and compares the HCM and Federal Highway Administration (FHWA) vehicle classification schemes. FHWA Classifications 4 and 5 are treated as SUTs by the HCM, while FHWA Classifications 6 and higher are considered as TTs.

Tractor-trailers are also sometimes referred to as combination trucks.

Two distinct methodologies are offered to assess the effect of heavy vehicles on capacity and LOS on freeways in the HCM:

1. Traditional passenger car equivalency (PCE) factors that allow the analyst to convert a mixed stream of cars and trucks to a single uniform PCE stream for purpose of analysis; and
2. A mixed-flow model that directly assesses the capacity, speed, and density of traffic streams that include a significant percentage of heavy vehicles operating on a single or composite grade.

This chapter's core methodology uses the PCE approach, while the mixed-flow model is presented in Volume 4 as an extension of the methodology. The mixed-flow model for single grades is found in Chapter 26, Freeway and Highway Segments: Supplemental, while the model for composite grades is found in Chapter 25, Freeway Facilities: Supplemental. The mixed-flow model form is fully consistent with Equation 12-1 and uses supporting equations to estimate a SAF, CAF, breakpoint, density at capacity, speed at capacity, and exponent calibration parameter. When the mixed-flow models are used, no PCEs are needed, since the passenger car, SUT, and TT volumes are used directly in the estimation of mixed-flow speed and density.

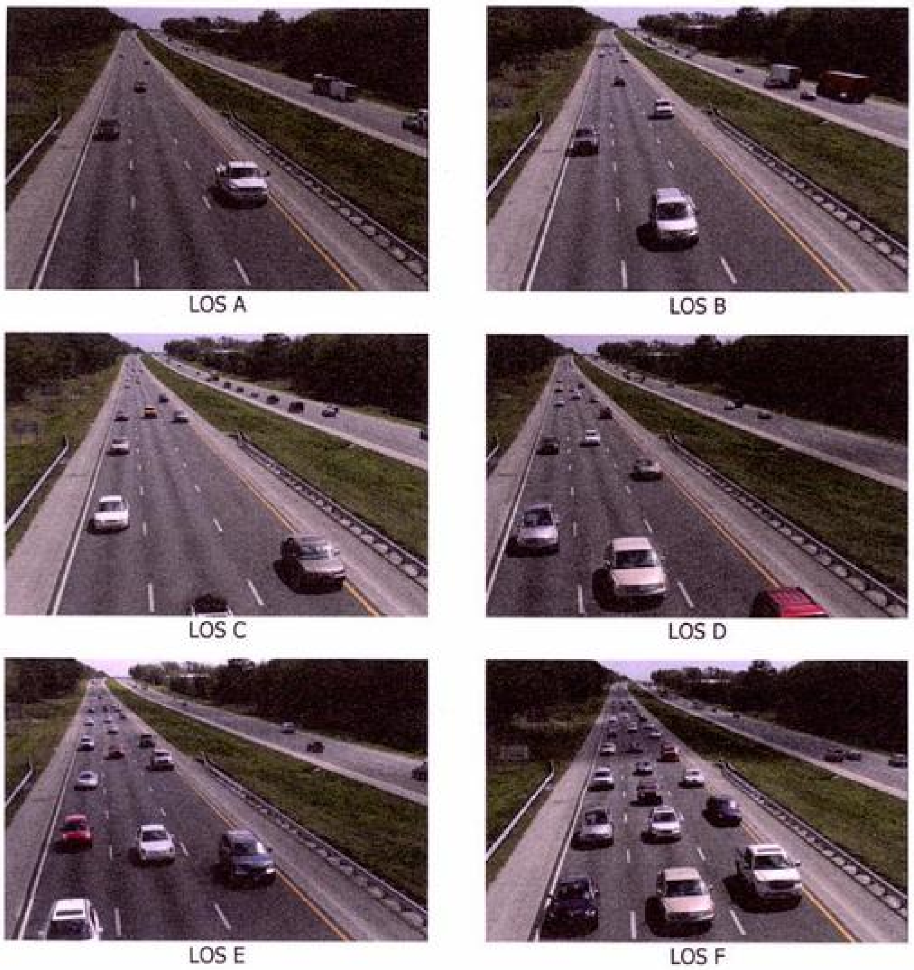
In fact, the mixed-flow method was used to generate the PCE tables as well as an equation for estimating the PCE value for any traffic mix of SUTs and TTs, as shown in Section 3. These PCE tables, and the associated equations in Volume 4, can be used to assess the LOS for a given mixed-flow segment without the direct use of the mixed-flow model. The PCE values are predicated on equivalency between the mixed-flow rate at capacity (in vehicles per hour per lane) and the flow rate of the equivalent automobile-only traffic stream (in passenger cars per hour per lane). The PCE tables assume the following splits between SUTs and TTs: 30% SUTs and 70% TTs, 50% SUTs and 50% TTs, and 70% SUTs and 30% TTs. The PCE equation on which the tables are based allows other truck mixes to be assessed.

If the PCE tables are used by themselves, the resulting speeds and densities for the equivalent automobile-only traffic stream may differ from those characterizing the mixed-flow condition. For most freeway analyses, PCE tables are sufficient and provide a reasonable approximation of the truck effects. However, if truck percentages are high or grades are significant, the mixed-flow model is expected to give a more accurate result. If estimates of the actual mixed-flow speeds and densities are desired, the mixed-flow model in Volume 4 should be used. If the basic freeway segment is analyzed as part of a freeway facility with the methodology in Chapter 10, a PCE approximation is typically appropriate and recommended.

LEVEL OF SERVICE

LOS on basic freeway and multilane highway segments is defined by density. Although speed is a major concern of drivers related to service quality, describing LOS on the basis of speed would be difficult, since it remains constant up to high flow rates [i.e., 1,000 to 1,800 pc/h/ln for basic freeway segments (depending on the FFS) and 1,400 pc/h/ln for multilane highway segments]. Density describes a motorist's proximity to other vehicles and is related to a motorist's freedom to maneuver within the traffic stream. Unlike speed, density is sensitive to flow rates throughout the range of flows. Exhibit 12-14 illustrates the six levels of service defined for basic freeway segments.

Exhibit 12-14
LOS Examples for Basic Freeway Segments



LOS Described

LOS A describes free-flow operations. FFS prevails on the freeway or multilane highway, and vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream. The effects of incidents or point breakdowns are easily absorbed.

LOS B represents reasonably free-flow operations, and FFS on the freeway or multilane highway is maintained. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents are still easily absorbed.

LOS C provides for flow with speeds near the FFS of the freeway or multilane highway. Freedom to maneuver within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service quality will be significant. Queues may be expected to form behind any significant blockages.

LOS D is the level at which speeds begin to decline with increasing flows, with density increasing more quickly. Freedom to maneuver within the traffic stream is seriously limited, and drivers experience reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.

LOS E describes operation at or near capacity. Operations on the freeway or multilane highway at this level are highly volatile because there are virtually no usable gaps within the traffic stream, leaving little room to maneuver within the traffic stream. Any disruption to the traffic stream, such as vehicles entering from a ramp or an access point or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic stream. Toward the upper boundary of LOS E, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown and substantial queuing. The physical and psychological comfort afforded to drivers is poor.

Oversaturated conditions are represented by LOS F.

LOS F describes unstable flow. Such conditions exist within queues forming behind bottlenecks. Breakdowns occur for a number of reasons:

- Traffic incidents can temporarily reduce the capacity of a short segment, so that the number of vehicles arriving at a point is greater than the number of vehicles that can move through it.
- Points of recurring congestion, such as merge or weaving segments and lane drops, experience very high demand in which the number of vehicles arriving is greater than the number of vehicles that can be discharged.
- In analyses using forecast volumes, the projected flow rate can exceed the estimated capacity of a given location.

In all cases, breakdown occurs when the ratio of existing demand to actual capacity, or of forecast demand to estimated capacity, exceeds 1.00. LOS F operations within a queue are the result of a breakdown or bottleneck at a downstream point. In practical terms, the point of the breakdown has a *d/c* ratio greater than 1.00 and is also labeled LOS F, although actual operations at the breakdown point and immediately downstream may actually reflect LOS E conditions. Whenever queues due to a breakdown exist, they have the potential to extend upstream for considerable distances. In that case, the upstream conditions (in the queue) will likely operate at LOS F speeds and densities, even if the segment-level predictions are LOS E or better. Therefore, for accurate estimation of the operational performance of these queue spillback effects, a freeway facility analysis should be conducted by using the procedure in Chapter 10 whenever one or more segment demands exceed capacity.

LOS Criteria

A basic freeway or multilane highway segment can be characterized by three performance measures: density in passenger cars per mile per lane, space mean speed in miles per hour, and the ratio of demand flow rate to capacity (v/c). Each of these measures is an indication of how well traffic is being accommodated by the basic freeway segment.

Because speed is constant through a broad range of flows and the v/c ratio is not directly discernible to road users (except at capacity), the service measure for basic freeway and multilane highway segments is density. Exhibit 12-15 shows the criteria.

Exhibit 12-15
LOS Criteria for Basic Freeway and Multilane Highway Segments

LOS	Density (pc/mi/ln)
A	≤11
B	>11–18
C	>18–26
D	>26–35
E	>35–45
F	Demand exceeds capacity OR density > 45

The LOS thresholds for basic freeway and multilane highway segments are the same for urban and rural locations, as defined by the FHWA smoothed or adjusted urbanized boundaries (*8*). However, note that a freeway facilities analysis (Chapter 10) defines different LOS thresholds for urban and rural *facilities*.

For all levels of service, the density boundaries on basic freeway segments are the same as those for multilane highways. Traffic characteristics are such that the maximum flow rates at any given LOS are lower on multilane highways than on similar basic freeway segments.

The specification of maximum densities for LOS A to D is based on the collective professional judgment of the members of the Transportation Research Board's Committee on Highway Capacity and Quality of Service. The upper value shown for LOS E (45 pc/mi/ln) is the maximum density at which sustained flows at capacity are expected to occur. In effect, as indicated in the speed–flow curves of Exhibit 12-7, when a density of 45 pc/mi/ln is reached, flow is at capacity, and the v/c ratio is 1.00.

In the application of this chapter's methodology, however, LOS F is identified when demand exceeds capacity because the analytical methodology *does not allow* the determination of density when demand exceeds capacity. Although the density will be greater than 45 pc/h/ln, the methodology of Chapter 10, Freeway Facilities Core Methodology, must be applied to determine a more precise density for such cases.

Exhibit 12-16 illustrates the range of densities for a given LOS on the base speed–flow curves for basic freeway segments. On a speed–flow plot, density is a line of constant slope starting at the origin. The LOS boundaries were defined to produce reasonable ranges for each LOS letter. Exhibit 12-17 shows the same relationships applied to multilane highway segments. The two dashed lines in the latter exhibit correspond to speed–flow relationships that were extrapolated from other results but that have not been calibrated from field data.

Exhibit 12-16
LOS Criteria and Speed–Flow Curves for Basic Freeway Segments

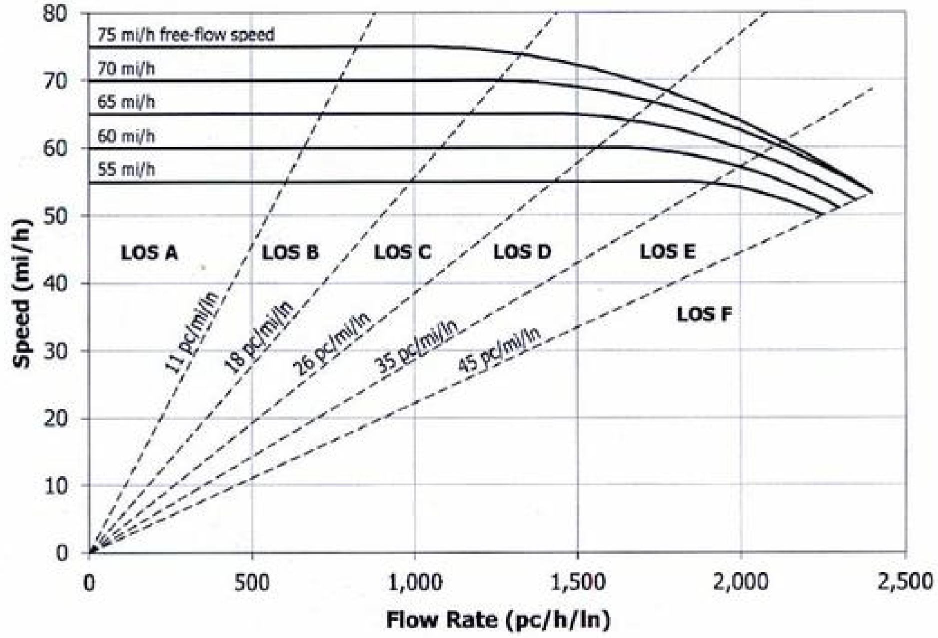
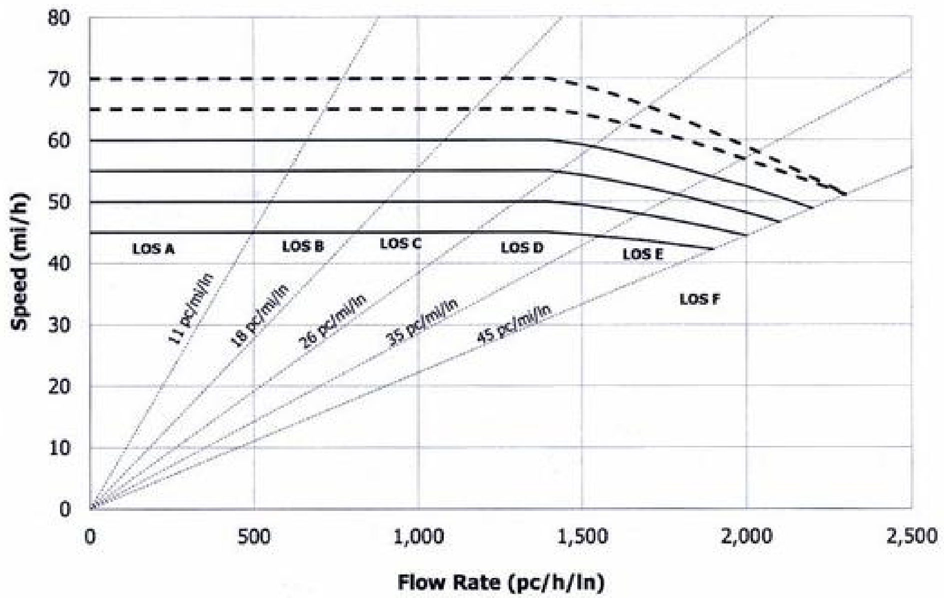


Exhibit 12-17
LOS Criteria and Speed–Flow Curves for Multilane Highway Segments



Note: Dashed curves are extrapolated and not based on field data.

3. MOTORIZED VEHICLE CORE METHODOLOGY

This chapter's methodology can be used to analyze the capacity, LOS, and lane requirements of basic freeway or multilane highway segments and the effects of design features on their performance. The methodology is based on the results of an NCHRP study (*4*), which has been partially updated (5). A number of significant publications were also used in the development of the methodology (6, 7, *9*–17).

SCOPE OF THE METHODOLOGY

The methodology described in this section is applicable to general purpose uninterrupted-flow, undersaturated basic freeway and multilane segments. Oversaturated conditions on basic freeway segments can be analyzed with the method described in Chapter 10, Freeway Facilities Core Methodology. Extensions of the methodology described in Section 4 address basic managed lane segments and bicycle LOS on multilane highways. Chapter 26, Freeway and Highway Segments: Supplemental, presents a method to analyze freeway operations on segments with significant truck presence, a prolonged single upgrade, or both.

Spatial and Temporal Limits

Determining capacity or LOS requires uniform traffic and roadway conditions on the analysis segment. Thus, any point where roadway or traffic conditions change must mark a boundary of the analysis segment.

At every ramp–freeway (or ramp–multilane highway) junction, the demand volume changes as some vehicles enter or leave the traffic stream. Thus, any ramp junction should mark a boundary between adjacent basic freeway or multilane highway segments.

In addition to ramp–freeway junctions, the following conditions generally dictate that a boundary be established between basic freeway or multilane highway segments:

Ramp junctions, grade changes of 2% or more, changes in the freeway's geometric characteristics, and changes in speed limit are some of the conditions dictating establishment of basic freeway segment or multilane highway boundaries.

- Change in the number of lanes (cross section);
- Changes in lane width or lateral clearance;
- Grade change of 2% or more on a specific or composite grade;
- Change in terrain category (for general terrain segments);
- Presence of a traffic signal, STOP sign, or roundabout along a multilane highway;
- Significant change in the access point density or total ramp density;
- Presence of a bottleneck condition;
- Change in posted speed limit; or
- Presence of an access point at which a significant number or percentage of vehicles enters or leaves a multilane highway.

The last item in this list is not directly involved in the analysis of a basic freeway or multilane highway segment but would probably reflect changes in ramp or access point density or other features.

The analysis period for any freeway or multilane highway analysis is generally the peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

If demand volumes are used, demand flow rates are estimated through use of the peak hour factor (PHF). When 15-min volumes are measured directly, the analysis period within the hour that has the highest volumes is selected, and flow rates are the 15-min volumes multiplied by 4. For subsequent computations in the methodology, the PHF is set to 1.00.

Performance Measures

The core motorized vehicle methodology generates the following performance measures:

- Capacity,
- FFS,
- Demand- and volume-to-capacity ratios,
- Space mean speed,
- Average density, and
- Motorized vehicle LOS.

Limitations of the Methodology

This chapter's methodologies for basic freeway segments and multilane highways do not apply to or take into account (without modification by the analyst) the following:

- Lane controls (to restrict lane changing);
- Extended bridge and tunnel segments;
- Segments near a toll plaza;
- Facilities with a FFS more than 75 mi/h for basic freeway segments or more than 70 mi/h for multilane highways;
- Facilities with a base FFS less than 55 mi/h for freeways and less than 45 mi/h for multilane highways, although lower FFS values can be achieved for freeway segments by calibrating a SAF;
- Posted speed limit and enforcement practices;
- Presence of intelligent transportation systems (ITS) related to vehicle or driver guidance;
- Capacity-enhancing effects of ramp metering;
- The influence of downstream queuing on a segment;
- Operational effects of oversaturated conditions; and
- Operational effects of construction operations.

Active traffic and demand management measures for freeways discussed in Chapter 37 consist of the following:

- *Ramp metering,*
- *Congestion pricing,*
- *Traveler information systems,*
- *Dynamic lane and shoulder management,*
- *Speed harmonization,*
- *Incident management, and*
- *Work zone traffic management.*

Many of these strategies can be evaluated with methodologies in Chapter 11.

The last four items in the list of limitations above are addressed in a freeway facility analysis context, as described in Chapter 10. The following are additional limitations for this chapter's multilane highway methodology:

- The effect of lane drops and lane additions at the beginning or end of multilane highway segments;
- Possible queuing impacts when a multilane highway segment transitions to a two-lane highway segment;
- The negative impacts of poor weather conditions, traffic accidents or incidents, railroad crossings, or construction operations on multilane highways;
- Differences between various types of median barriers and the difference between the impacts of a median barrier and a TWLTL;
- Significant presence of on-highway parking;
- Presence of bus stops that have significant use; and
- Significant pedestrian activity.

The last three factors are more representative of an urban or suburban arterial, but they may also exist on multilane highway facilities with more than 2 mi between traffic signals. When these factors are present on uninterrupted-flow segments of multilane highways, the methodology does not deal with their impact on flow. In addition, this methodology cannot be applied to highways with a total of three lanes in both directions, which should be analyzed as two-lane highways with periodic passing lanes by using the methods of Chapter 15.

Uninterrupted-flow multilane highway facilities that allow access solely through a system of on-ramps and off-ramps from grade separations or service roads should be analyzed as freeways. Note that some ramp access or egress points may be present on a multilane highway where most access or egress points are at-grade junctions of some type.

Uninterrupted-flow multilane highway facilities that allow access solely through a system of on-ramps and off-ramps from grade separations or service roads should be analyzed as freeways.

To address most of the limitations listed above, the analyst would have to utilize alternative tools or draw on other research information and develop special-purpose modifications of this methodology. Operational effects of oversaturated conditions, incidents, work zones, and weather and lighting conditions can be evaluated with the methodology of Chapter 10 and adjustment factors for capacity and FFS found in Chapter 11. Operational effects of active traffic and demand management (ATDM) measures can be evaluated by using the procedures in Chapter 11, Freeway Reliability Analysis. A broader overview of ATDM strategies is presented in Chapter 37, ATDM: Supplemental.

Alternative Tools

Strengths of HCM Procedures

This chapter's procedures were developed on the basis of extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent an expert consensus.

The HCM methodology provides FFS as an output, incorporates geometric characteristics, provides explicit capacity estimates, and produces a single deterministic estimate of traffic density.

Specific strengths of the HCM basic freeway and multilane highway segment methodology include the following:

OVERVIEW OF THE METHODOLOGY

Exhibit 12-19 illustrates the basic methodology used in operational analysis. The methodology can also be directly applied to determine the number of lanes required to provide a target LOS for a given demand volume.

Exhibit 12-19
Overview of Operational Analysis Methodology for Basic Freeway and Multilane Highway Segments

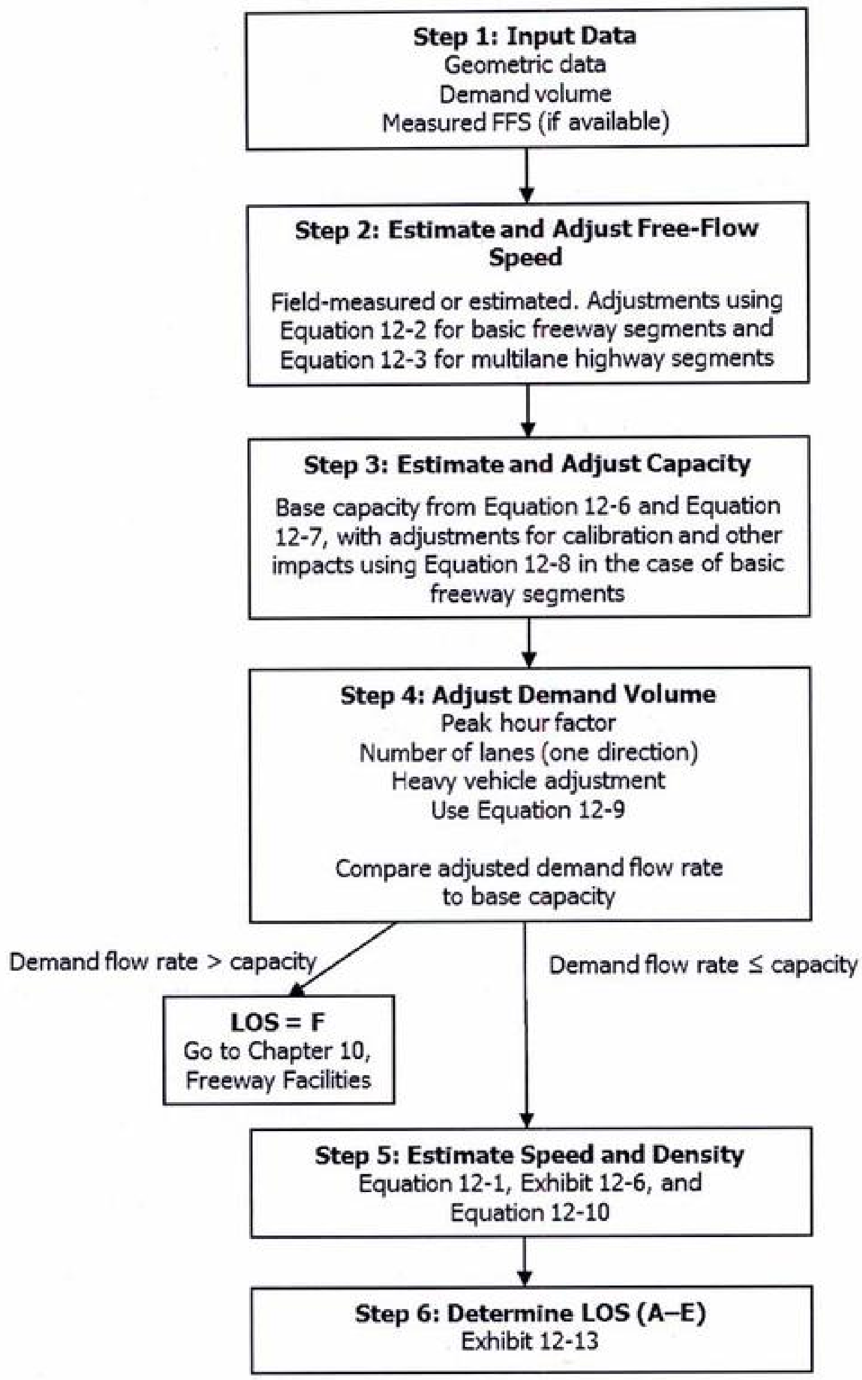


Exhibit 12-19 illustrates the methodology for an operational analysis. Other types of analyses are described in Section 5, Applications.

COMPUTATIONAL STEPS

Step 1: Input Data

For a typical operational analysis, as noted previously, the analyst would have to specify (with either site-specific or default values) the demand volume; number and width of lanes; right-side or overall lateral clearance; total ramp or access point density; percent of heavy vehicles; PHF; terrain; and the driver population, speed, and capacity adjustment factors (if necessary).

Step 2: Estimate and Adjust FFS

FFS can be determined directly from field measurements or can be estimated as described below. Statement of FFS in 5-mi/h increments is no longer necessary. This change is important in accounting for the effect of weather or work zones, which may reduce the value of the base FFS.

Field Measurement of FFS

FFS is the mean speed of passenger cars during periods of low to moderate flow.

FFS is the mean speed of passenger cars measured during periods of low to moderate flow (up to 500 pc/h/ln). For a specific freeway or multilane highway segment, average speeds are virtually constant in this range of flow rates. Field measurement of FFS, if possible, is preferable. If the FFS is measured directly, no adjustments are applied to the measured value.

Some freeways may have lower posted speed limits for trucks, which may affect the mixed-flow FFS. In these cases, field studies are recommended, since the FFS estimation methodology below is not sensitive to the posted speed limit or the presence of a high percentage of trucks.

The speed study should be conducted at a location that is representative of the segment at a time when flow rates are less than 1,000 pc/h/ln. The speed study should measure the speeds of all passenger cars or use a systematic sample (e.g., every 10th car in each lane). A sample of at least 100 passenger car speeds should be obtained. Any speed measurement technique that has been found acceptable for other types of traffic engineering applications may be used. Further guidance on the conduct of speed studies is provided in standard traffic engineering publications, such as the *Manual of Transportation Engineering Studies* (16).

Estimating FFS

Basic Freeway Segments

Field measurements for future facilities are not possible, and field measurement may not be possible or practical for all existing facilities. In such cases, the segment's FFS may be estimated by using Equation 12-2, which is based on the physical characteristics of the segment under study:

$$FFS = BFFS - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

Equation 12-2

where

FFS = free-flow speed of the basic freeway segment (mi/h);

$BFFS$ = base FFS for the basic freeway segment (mi/h);

f_{LW} = adjustment for lane width, from Exhibit 12-20 (mi/h);

f_{RLC} = adjustment for right-side lateral clearance, from Exhibit 12-21 (mi/h); and

TRD = total ramp density (ramps/mi).

Multilane Highway Segments

For multilane highway segments, the FFS can be estimated by using Equation 12-3, which is based on the physical characteristics of the segment under study. It is evident that while the base FFS and the lane width adjustment are shared with the estimation method for basic freeway segments in Equation 12-2, the remaining terms are unique to multilane highway segments:

Equation 12-3

$$FFS = BFFS - f_{LW} - f_{TLC} - f_M - f_A$$

where

FFS = free-flow speed of the multilane highway segment (mi/h);

$BFFS$ = base FFS for the multilane highway segment (mi/h);

f_{LW} = adjustment for lane width, from Exhibit 12-20 (mi/h);

f_{TLC} = adjustment for total lateral clearance, from Exhibit 12-22 (mi/h);

f_M = adjustment for median type, from Exhibit 12-23 (mi/h); and

f_A = adjustment for access point density, from Exhibit 12-24 (mi/h).

Adjustments to FFS

Base FFS

This methodology covers basic freeway segments with a FFS in the range of 55 to 75 mi/h. The predictive algorithm for FFS therefore starts with a value greater than 75 mi/h, specifically a default base FFS of 75.4 mi/h, which resulted in the most accurate predictions in the underlying research.

The methodology covers multilane highway segments with a FFS in the range of 45 to 70 mi/h. The most significant value in Equation 12-3 is *BFFS*. There is not a great deal of information available to help establish a base value. In one sense, it is like the design speed—it represents the potential FFS based only on the highway's horizontal and vertical alignment, not including the impacts of lane widths, lateral clearances, median type, and access points. The design speed may be used for *BFFS* if it is available.

Although speed limits are not always uniformly set, *BFFS* for multilane highways may be estimated, if necessary, as the posted or statutory speed limit plus 5 mi/h for speed limits 50 mi/h and higher and as the speed limit plus 7 mi/h for speed limits less than 50 mi/h.

Adjustment for Lane Width

The base condition for lane width is 12 ft or greater. When the average lane width across all lanes is less than 12 ft, the FFS is negatively affected. Adjustments to reflect the effect of narrower average lane width are shown in Exhibit 12-20.

Average Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Exhibit 12-20
Adjustment to FFS for Average Lane Width for Basic Freeway and Multilane Highway Segments

Adjustment for Right Lateral Clearance on Freeway Segments

The base condition for right-side lateral clearance is 6 ft or greater. The lateral clearance is measured from the right edge of the travel lane to the nearest lateral obstruction. Care must be taken in identifying a "lateral obstruction." Some obstructions may be continuous, such as retaining walls, concrete barriers, guardrails, or barrier curbs. Others may be periodic, such as light supports or bridge abutments. In some cases, drivers may become accustomed to certain types of obstructions, and their influence on traffic is often negligible.

Exhibit 12-21 shows the adjustment to FFS due to the existence of obstructions closer than 6 ft from the right travel lane edge. Median clearances of 2 ft or more on the left side of the travel lanes generally have little impact on traffic. No adjustments are available to reflect the presence of left-side lateral obstructions closer than 2 ft from the left travel lane edge. Such situations are rare on modern freeways, except in constrained work zones.

Right-Side Lateral Clearance (ft)	Lanes in One Direction			
	2	3	4	≥5
≥6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

Note: Interpolate for noninteger values of right-side lateral clearance.

Exhibit 12-21
Adjustment to FFS for Right-Side Lateral Clearance, f_{RLC} (mi/h), for Basic Freeway Segments

The impact of a right-side lateral clearance restriction depends on both the distance to the obstruction and the number of lanes in one direction on the basic freeway segment. A lateral clearance restriction causes vehicles in the right lane to move somewhat to the left. This movement, in turn, affects vehicles in the next lane. As the number of lanes increases, the overall effect on freeway operations decreases.

Adjustment for Total Lateral Clearance on Multilane Highway Segments

Clearance restrictions on either the right or the left side of the highway reduce the FFS.

The adjustment for total lateral clearance (TLC) on multilane highway segments is based on TLC at the roadside (right side) and at the median (left side). Fixed obstructions with lateral clearance effects include light standards, signs, trees, abutments, bridge rails, traffic barriers, and retaining walls. Standard raised curbs are not considered to be obstructions.

Right-side lateral clearance is measured from the right edge of the travel lanes to the nearest periodic or continuous roadside obstruction. If such obstructions are farther than 6 ft from the edge of the pavement, a value of 6 ft is used.

Left-side lateral clearance is measured from the left edge of the travel lanes to the nearest periodic or continuous obstruction in the median. If such obstructions are farther than 6 ft from the edge of the pavement, a value of 6 ft is used.

Use 6 ft as the left-side clearance for undivided highways and highways with TWLTLs.

Left-side lateral clearances are subject to some judgment. Many types of common median barriers do not affect driver behavior if they are no closer than 2 ft from the edge of the travel lane, including concrete and W-beam barriers. A value of 6 ft would be used in such cases. Also, when the multilane highway segment is undivided or has a TWLTL, no left-side lateral clearance restriction is assumed, and a value of 6 ft is applied. A separate adjustment, described next, accounts for the impact of an undivided highway on FFS.

Equation 12-4 is used to determine TLC:

Equation 12-4

$$TLC = LC_R + LC_L$$

where

TLC = total lateral clearance (ft) (maximum value 12 ft),

LC_R = right-side lateral clearance (ft) (maximum value 6 ft), and

LC_L = left-side lateral clearance (ft) (maximum value 6 ft).

Exhibit 12-22 shows the reduction in FFS due to lateral obstructions on the multilane highway.

Exhibit 12-22
Adjustment to FFS for Lateral Clearances for Multilane Highways

Four-Lane Highways		Six-Lane Highways	
TLC (ft)	Reduction in FFS, f_{LC} (mi/h)	TLC (ft)	Reduction in FFS, f_{LC} (mi/h)
12	0.0	12	0.0
10	0.4	10	0.4
8	0.9	8	0.9
6	1.3	6	1.3
4	1.8	4	1.7
2	3.6	2	2.8
0	5.4	0	3.9

Note: Interpolation to the nearest 0.1 is recommended.

Adjustment for Type of Median on Multilane Highways

The FFS is reduced on undivided highways.

The adjustment for type of median is given in Exhibit 12-23. Undivided multilane highways reduce the FFS by 1.6 mi/h.

Exhibit 12-23
Adjustment to FFS for Median Type for Multilane Highways

Median Type	Reduction in FFS, f_M (mi/h)
Undivided	1.6
TWLTL	0.0
Divided	0.0

Adjustment for Total Ramp Density on Basic Freeway Segments

Equation 12-2 includes a term that accounts for the impact of total ramp density on FFS. Total ramp density is defined as the number of ramps (on and off, one direction) located between 3 mi upstream and 3 mi downstream of the midpoint of the basic freeway segment under study, divided by 6 mi. The total ramp density has been found to be a measure of the impact of merging and diverging vehicles on FFS.

Adjustment for Access Point Density on Multilane Highway Segments

FFS is reduced as the access point density increases.

Exhibit 12-24 presents the adjustment to FFS for various levels of access point density. Studies indicate that for each access point per mile, the estimated FFS decreases by approximately 0.25 mi/h, regardless of the type of median.

The number of access points per mile is determined by dividing the total number of access points (i.e., driveways and unsignalized intersections) on the right side of the highway in the direction of travel by the length of the segment in miles. An intersection or driveway should only be included in the count if it influences traffic flow. Access points that go unnoticed by drivers or that have little activity should not be used to determine access point density.

Exhibit 12-24
Adjustment to FFS for Access Point Density for Multilane Highways

Access Point Density (access points/mi)	Reduction in FFS, f_A (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
≥40	10.0

Note: Interpolation to the nearest 0.1 is recommended.

Although the calibration of this adjustment did not include one-way multilane highway segments, inclusion of intersection approaches and driveways on both sides of the facility might be appropriate in determining the access point density on one-way segments.

Speed Adjustment Factor for Basic Freeway Segments

The estimated FFS for basic freeway segments can be further adjusted to reflect, for example, effects of inclement weather. In this case, an adjusted free-flow speed FFS_{adj} is calculated by multiplying the FFS by a SAF as shown in Equation 12-5:

Equation 12-5

$$FFS_{adj} = FFS \times SAF$$

where *SAF* is the speed adjustment factor. The speed adjustment factor can represent a combination of sources, including weather and work zone effects. Default speed adjustment factors and guidance for how to apply them are found in Chapter 11.

The SAF may also be used to calibrate the estimated FFS for local conditions or other effects that contribute to a reduction in FFS. For example, poor pavement conditions or sun glare may cause drivers to reduce their speeds even under low-volume conditions. The adjusted FFS can be used directly in the speed–flow relationship for basic freeway segments in Exhibit 12-6 to define a continuous speed–flow curve that explicitly considers this adjusted FFS. Finally, the effect of unfamiliar drivers on FFS can also be accounted for by using an adjusted FFS. While the driver population SAF defaults to 1.0 in the base procedure, general guidance for selecting an appropriate SAF to account for this factor is given in Section 4 of Chapter 26.

No adjustment of the speed–flow equation using these SAFs is possible for multilane highway segments, since no empirical research exists for applying these effects on multilane highways.

Step 3: Estimate and Adjust Capacity

In this step, the base capacity for the basic freeway or multilane highway segment is estimated. The segment capacity is principally a function of the segment FFS, but it can be adjusted to calibrate the segment for local conditions or to reflect impacts of adverse weather conditions, incidents, or other factors. The base capacity values for basic freeway segments and multilane highway segments are listed in Exhibit 12-4 for various values of FFS. Because of the ability to interpolate between different FFS values, the resulting segment capacities should also be interpolated. Alternatively, the base capacity c for a basic freeway segment (in passenger cars per hour per lane) can be estimated directly with Equation 12-6, while the base capacity for a multilane highway segment can be estimated directly with Equation 12-7:

Equation 12-6

$$c \text{ (basic freeway segment)} = 2{,}200 + 10 \times (FFS_{adj} - 50)$$

Equation 12-7

$$c \text{ (multilane highway segment)} = 1{,}900 + 20 \times (FFS_{adj} - 45)$$

where all variables have been previously defined.

The capacities resulting from application of these equations can never exceed the base capacities listed in Exhibit 12-4, which are 2,400 pc/h/ln for basic freeway segments and 2,300 pc/h/ln for multilane highway segments. Similarly, the FFS used in these equations should not exceed 75 mi/h for basic freeway segments or 70 mi/h for multilane highway segments.

Adjustment to Capacity for Local Calibration

The base capacities estimated by using Equation 12-6 and Equation 12-7 are based on ideal conditions and are expressed in units of passenger cars per hour per lane. The presence of a significant proportion of heavy vehicles, especially in combination with grades, will result in a net decrease in the observed capacity when converted to units of vehicles per hour per lane. As a result, sensor-based measurements of freeway capacities (in vehicles per hour per lane) may be significantly less than the base values stated above.

Many factors other than heavy vehicle effects can contribute to a reduction in basic freeway segment capacity. Examples of capacity-reducing effects include the following:

- Capacity adjustment for driver population, which is intended to account for the level of unfamiliar drivers in the traffic stream (see Section 4 of Chapter 26 for additional details);
- Turbulence generated from lane drops between two basic segments;
- Turbulence due to merging, diverging, or weaving maneuvers between two basic segments;
- Capacity reductions due to poor sight distance—for example, due to crest vertical curves or horizontal curves;
- Narrow lane widths or low lateral clearances in addition to the effects on FFS presented in Step 2;
- Travel through tunnels or across bridges;
- Poor pavement conditions; and

- Friction effects due to roadside features and attractions that cause drivers to increase following headways.

In these cases, development of a local estimate of capacity and use of that estimate to calibrate a CAF for the segment under study are highly recommended. In the absence of generalized national data on these capacity-reducing effects, a local calibration study or expert judgment is needed to produce a reasonable estimate of segment performance. A methodology for estimating freeway capacities from sensor data is provided in Section 5 of Chapter 26.

Adjustment to Capacity for Basic Freeway Segments

The capacity of a basic freeway segment may be adjusted further to account for the impacts of adverse weather, driver population, occurrence of traffic incidents, or a combination of such influences. The methodology for making these adjustments is the same as that for other types of freeway segments. CAF defaults are found in Chapter 11, along with additional discussion on how to apply them. For convenience, a brief summary is provided here.

The capacity of a basic freeway segment can be adjusted as shown in Equation 12-8:

$$c_{adj} = c \times CAF$$

Equation 12-8

where

c_{adj} = adjusted capacity of segment (pc/h),

c = base capacity of segment (pc/h), and

CAF = capacity adjustment factor (unitless).

The CAF can have several components, including weather, incident, work zone, driver population, and calibration adjustments. The adjustments for weather and incidents are most commonly applied in the context of a reliability analysis as described in Chapter 11, Freeway Reliability Analysis. If desired, capacity can be adjusted further to account for unfamiliar drivers in the traffic stream. While the default CAF for this effect is set to 1.0, guidance is provided in Section 4 of Chapter 26, where estimates for the CAF based on the composition of the driver population are provided.

No adjustment of the speed–flow equation using these CAFs is possible for multilane highway segments, since no empirical research exists for applying these effects to multilane highways.

Step 4: Adjust Demand Volume

Since the speed–flow curves and parameters of Exhibit 12-6 are based on flow rates in equivalent passenger cars per hour on the basic freeway segment, demand volumes expressed as vehicles per hour under prevailing conditions must be converted to this basis by using Equation 12-9:

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

Equation 12-9

where

v_p = demand flow rate under equivalent base conditions (pc/h/ln),

V = demand volume under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

N = number of lanes in analysis direction (ln), and

f_{HV} = adjustment factor for presence of heavy vehicles (decimal).

Peak Hour Factor

The PHF represents the variation in traffic flow within an hour. Observations of traffic flow consistently indicate that the flow rates found in the peak 15 min within an hour are not sustained throughout the entire hour. The application of the PHF in Equation 12-9 accounts for this phenomenon.

On freeways, typical PHFs range from 0.85 to 0.98 (18). On multilane highways, typical PHFs range from 0.75 to 0.95. Lower values within that range are typical of lower-volume conditions. Higher values within that range are typical of urban and suburban peak-hour conditions. Field data should be used if possible to develop PHFs that represent local conditions.

Adjustment for Heavy Vehicles

All heavy vehicles are classified as SUTs or TTs. Recreational vehicles and buses are treated as SUTs. The heavy vehicle adjustment factor f_{HV} is computed from the combination of the two heavy vehicle classes, which are added to get an overall truck percentage P_T.

Equation 12-10

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

f_{HV} = heavy vehicle adjustment factor (decimal),

P_T = proportion of SUTs and TTs in traffic stream (decimal), and

E_T = passenger car equivalent of one heavy vehicle in the traffic stream (PCEs).

The adjustment factor is found in a two-step process. First, the PCE for each truck is found for the prevailing conditions under study. These equivalency values represent the number of passenger cars that would use the same amount of freeway capacity as one truck under the prevailing conditions. Second, Equation 12-10 is used to convert the PCE values to the adjustment factor.

The effect of heavy vehicles on traffic flow depends on the terrain and grade conditions on the segment as well as traffic composition. PCEs can be selected for one of two conditions:

- Extended freeway and multilane highway segments in general terrain, or
- Specific upgrades or downgrades.

Each of these conditions is more precisely defined and discussed below. However, research has shown that PCEs should be used mostly for addressing capacity and LOS issues. They provide reasonable results for speeds and densities when the grade is slight or the truck percentage is low. For combinations that include steep grades, high truck percentages, or both, the mixed-flow model described in Chapter 25 (for composite grades) and Chapter 26 (for single grades) is recommended for computing mixed-flow speeds and densities and automobile and truck speeds in the mixed traffic stream.

Equivalents for General Terrain Segments

General terrain refers to extended lengths of freeway and multilane highways containing a number of upgrades and downgrades where no one grade is long enough or steep enough to have a significant impact on the operation of the overall segment. General terrain can be either level or rolling. To determine which of these terrain types applies, each upgrade and downgrade should be considered to be a single grade, even if the grade is not uniform. The total length of the upgrade or downgrade is used with the steepest grade it contains. The categorization of a segment as having either level or rolling terrain is as follows:

- *Level terrain*: Any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars. This type of terrain typically contains short grades of no more than 2%.
- *Rolling terrain*: Any combination of grades and horizontal or vertical alignment that causes heavy vehicles to reduce their speed below those of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length.

No PCE is provided for mountainous terrain, which is any combination of grades and horizontal and vertical alignment that causes heavy vehicles to operate at crawl speed for significant distances or at frequent intervals. In this case, the mixed-flow model presented in Chapters 25 and 26 must be used to estimate speeds and densities. Exhibit 12-25 gives PCEs for the default mix of trucks under level and rolling terrain conditions.

Exhibit 12-25
PCEs for General Terrain Segments

Passenger Car Equivalent	Terrain Type	
	Level	Rolling
E_T	2.0	3.0

Equivalents for Specific Upgrades

Freeway and multilane highway segments longer than 0.5 mi with grades between 2% and 3% or longer than 0.25 mi with grades of 3% or greater should be considered as separate segments. Research (*19*) has revealed that the SUT population on freeways has a median weight-to-horsepower ratio of about 100 lb/hp while the TT population has a median weight-to-horsepower ratio of 150 lb/hp. These values can vary from one setting to another.

Exhibit 12-26 gives specific-segment PCE values for a 30%/70% SUT/TT mix, Exhibit 12-27 gives PCE values for a 50%/50% mix, and Exhibit 12-28 gives PCE values for a 70%/30% mix. The 30% SUT condition occurs more frequently on rural facilities; the 50% condition occurs more frequently on urban facilities. Exhibit 12-28 is recommended for conditions where the majority of the trucks in the traffic stream are SUTs. Note that for the exhibits, segment lengths for grades above 3.5% are limited to 1 mi, because steeper grades are rarely longer than this in practice.

Exhibit 12-26
PCEs for a Mix of 30% SUTs and 70% TTs

% Grade	Length (mi)	Percentage of Trucks (%)								
		2%	4%	5%	6%	8%	10%	15%	20%	>25%
-2	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.625	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.875	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.25	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.5	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
0	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.625	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.875	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.25	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	1.5	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
2	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	3.76	2.96	2.78	2.65	2.48	2.38	2.22	2.14	2.09
	0.625	4.47	3.33	3.08	2.91	2.68	2.54	2.34	2.23	2.17
	0.875	4.80	3.50	3.22	3.03	2.77	2.61	2.39	2.28	2.21
	1.25	5.00	3.60	3.30	3.09	2.83	2.66	2.42	2.30	2.23
	1.5	5.04	3.62	3.32	3.11	2.84	2.67	2.43	2.31	2.23
2.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	4.11	3.14	2.93	2.78	2.58	2.46	2.28	2.19	2.13
	0.625	5.04	3.62	3.32	3.11	2.84	2.67	2.43	2.31	2.23
	0.875	5.48	3.85	3.51	3.27	2.96	2.77	2.50	2.36	2.28
	1.25	5.73	3.98	3.61	3.36	3.03	2.83	2.54	2.40	2.31
	1.5	5.80	4.02	3.64	3.38	3.05	2.84	2.55	2.41	2.32
3.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	4.88	3.54	3.25	3.05	2.80	2.63	2.41	2.29	2.22
	0.625	6.34	4.30	3.87	3.58	3.20	2.97	2.64	2.48	2.38
	0.875	7.03	4.66	4.16	3.83	3.39	3.12	2.76	2.57	2.46
	1.25	7.44	4.87	4.33	3.97	3.50	3.22	2.82	2.62	2.50
	1.5	7.53	4.92	4.38	4.01	3.53	3.24	2.84	2.63	2.51
4.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	5.80	4.02	3.64	3.38	3.05	2.84	2.55	2.41	2.32
	0.625	7.90	5.11	4.53	4.14	3.63	3.32	2.90	2.68	2.55
	0.875	8.91	5.64	4.96	4.50	3.92	3.56	3.07	2.82	2.67
	1	9.19	5.78	5.08	4.60	3.99	3.62	3.11	2.85	2.70
5.5	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	6.87	4.58	4.10	3.77	3.35	3.09	2.73	2.55	2.44
	0.625	9.78	6.09	5.33	4.82	4.16	3.76	3.21	2.93	2.77
	0.875	11.20	6.83	5.94	5.33	4.56	4.09	3.45	3.12	2.93
	1	11.60	7.04	6.11	5.47	4.67	4.18	3.51	3.17	2.97
6	0.125	2.62	2.37	2.30	2.24	2.17	2.12	2.04	1.99	1.97
	0.375	7.48	4.90	4.36	3.99	3.52	3.23	2.83	2.63	2.51
	0.625	10.87	6.66	5.79	5.21	4.46	4.01	3.39	3.08	2.89
	0.875	12.54	7.54	6.51	5.81	4.94	4.40	3.67	3.30	3.08
	1	13.02	7.78	6.71	5.99	5.07	4.51	3.75	3.37	3.14

Note: Interpolation in the exhibit is permitted.

Exhibit 12-27
PCEs for a Mix of 50% SUTs and 50% TTs

% Grade	Length (mi)	Percentage of Trucks (%) 2%	4%	5%	6%	8%	10%	15%	20%	>25%
-2	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.625	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.875	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.25	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.5	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
0	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.625	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.875	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.25	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	1.5	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
2	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	3.76	2.95	2.77	2.64	2.47	2.36	2.20	2.11	2.06
	0.625	4.32	3.24	3.01	2.84	2.63	2.49	2.29	2.19	2.12
	0.875	4.57	3.37	3.11	2.93	2.70	2.55	2.33	2.22	2.15
	1.25	4.71	3.45	3.17	2.99	2.74	2.58	2.36	2.24	2.17
	1.5	4.74	3.47	3.19	3.00	2.75	2.59	2.36	2.24	2.17
2.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	4.10	3.13	2.92	2.77	2.57	2.44	2.26	2.16	2.10
	0.625	4.84	3.52	3.23	3.03	2.77	2.61	2.38	2.26	2.18
	0.875	5.17	3.69	3.37	3.15	2.87	2.69	2.43	2.30	2.22
	1.25	5.36	3.79	3.45	3.22	2.92	2.73	2.47	2.33	2.24
	1.5	5.40	3.81	3.47	3.24	2.93	2.74	2.47	2.33	2.25
3.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	4.89	3.54	3.25	3.05	2.79	2.62	2.39	2.26	2.19
	0.625	6.05	4.15	3.75	3.47	3.11	2.89	2.58	2.42	2.32
	0.875	6.58	4.43	3.97	3.66	3.26	3.01	2.67	2.49	2.39
	1.25	6.88	4.58	4.10	3.77	3.35	3.09	2.72	2.53	2.42
	1.5	6.95	4.62	4.13	3.80	3.37	3.10	2.73	2.54	2.43
4.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	5.83	4.03	3.65	3.39	3.05	2.84	2.55	2.39	2.30
	0.625	7.53	4.92	4.38	4.01	3.53	3.24	2.83	2.62	2.50
	0.875	8.32	5.34	4.72	4.29	3.75	3.42	2.97	2.73	2.59
	1	8.53	5.45	4.81	4.37	3.81	3.47	3.00	2.76	2.62
5.5	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	6.97	4.63	4.14	3.81	3.38	3.11	2.74	2.55	2.43
	0.625	9.37	5.89	5.16	4.68	4.05	3.67	3.14	2.88	2.72
	0.875	10.49	6.48	5.65	5.09	4.37	3.93	3.34	3.03	2.85
	1	10.80	6.64	5.78	5.20	4.46	4.01	3.39	3.08	2.89
6	0.125	2.67	2.38	2.31	2.25	2.16	2.11	2.02	1.97	1.93
	0.375	7.64	4.98	4.43	4.05	3.56	3.26	2.85	2.64	2.51
	0.625	10.45	6.45	5.63	5.07	4.36	3.92	3.33	3.03	2.85
	0.875	11.78	7.16	6.20	5.56	4.74	4.24	3.56	3.22	3.01
	1	12.15	7.35	6.36	5.69	4.85	4.33	3.62	3.27	3.05

Note: Interpolation in the exhibit is permitted.

Exhibit 12-28
PCEs for a Mix of 70% SUTs and 30% TTs

% Grade	Length (mi)	Percentage of Trucks (%)								
		2%	4%	5%	6%	8%	10%	15%	20%	>25%
-2	0.125	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.375	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.625	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.875	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.25	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.5	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
0	0.125	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.375	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.625	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	0.875	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.25	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
	1.5	2.39	2.18	2.12	2.07	2.01	1.96	1.89	1.85	1.83
2	0.125	2.67	2.32	2.23	2.17	2.08	2.03	1.94	1.89	1.86
	0.375	3.63	2.82	2.64	2.52	2.35	2.25	2.10	2.02	1.97
	0.625	4.12	3.08	2.85	2.69	2.49	2.36	2.18	2.08	2.02
	0.875	4.37	3.21	2.96	2.78	2.56	2.42	2.22	2.11	2.05
	1.25	4.53	3.29	3.02	2.84	2.60	2.45	2.24	2.13	2.07
	1.5	4.58	3.31	3.04	2.86	2.61	2.46	2.25	2.14	2.07
2.5	0.125	2.75	2.36	2.27	2.20	2.11	2.04	1.95	1.90	1.87
	0.375	4.01	3.02	2.80	2.65	2.46	2.33	2.16	2.06	2.01
	0.625	4.66	3.35	3.08	2.88	2.64	2.48	2.26	2.15	2.08
	0.875	4.99	3.52	3.21	3.00	2.73	2.56	2.32	2.19	2.12
	1.25	5.20	3.64	3.30	3.08	2.79	2.60	2.35	2.22	2.14
	1.5	5.26	3.67	3.33	3.10	2.80	2.62	2.36	2.23	2.15
3.5	0.125	2.93	2.45	2.34	2.26	2.16	2.09	1.98	1.92	1.89
	0.375	4.86	3.46	3.16	2.96	2.69	2.53	2.30	2.18	2.10
	0.625	5.88	3.99	3.59	3.32	2.98	2.76	2.46	2.31	2.22
	0.875	6.40	4.26	3.81	3.51	3.12	2.88	2.55	2.38	2.28
	1.25	6.74	4.43	3.96	3.63	3.21	2.96	2.60	2.42	2.32
	1.5	6.83	4.48	3.99	3.66	3.24	2.98	2.62	2.44	2.33
4.5	0.125	3.13	2.56	2.43	2.34	2.21	2.13	2.01	1.95	1.91
	0.375	5.88	3.99	3.59	3.32	2.98	2.76	2.46	2.31	2.22
	0.625	7.35	4.75	4.22	3.85	3.39	3.10	2.71	2.51	2.39
	0.875	8.11	5.15	4.54	4.13	3.60	3.27	2.83	2.61	2.47
	1	8.33	5.27	4.63	4.21	3.66	3.33	2.87	2.64	2.50
5.5	0.125	3.37	2.69	2.53	2.42	2.28	2.19	2.05	1.98	1.94
	0.375	7.09	4.62	4.11	3.76	3.31	3.04	2.66	2.47	2.36
	0.625	9.13	5.68	4.97	4.49	3.88	3.51	3.00	2.74	2.59
	0.875	10.21	6.24	5.43	4.88	4.18	3.76	3.18	2.89	2.71
	1	10.52	6.41	5.57	5.00	4.27	3.83	3.24	2.93	2.75
6	0.125	3.51	2.76	2.59	2.47	2.32	2.22	2.08	2.00	1.95
	0.375	7.78	4.98	4.40	4.01	3.51	3.20	2.78	2.56	2.44
	0.625	10.17	6.23	5.42	4.87	4.17	3.75	3.18	2.88	2.71
	0.875	11.43	6.88	5.95	5.32	4.53	4.04	3.39	3.06	2.86
	1	11.81	7.08	6.11	5.46	4.64	4.13	3.45	3.11	2.90

Note: Interpolation in the exhibit is permitted.

The PCE values shown in this chapter have been estimated from simulation. They are also based on generalized analytical equations for the propulsion and resistance characteristics of SUTs and TTs (*19*). Different models based on more detailed vehicle dynamics simulators (e.g., *20, 21*) can produce different results. The PCEs establish an equivalency between the mixed-traffic capacity and the automobile-only capacity. The speeds associated with these PCE values are space mean speeds, and the densities are defined over the length of the segment. As noted previously, in evaluating composite grades, steep single grades, very high truck percentages, or a combination, the appropriate mixed-flow model from

Chapter 25 (composite grades) or Chapter 26 (single grades) is recommended in lieu of applying PCEs.

Check for LOS F

At this point, the demand volume has been converted to a demand flow rate in passenger cars per hour per lane under equivalent base conditions. This demand rate must be compared with the base capacity of the basic freeway or multilane highway segment (see Exhibit 12-4).

If demand exceeds capacity, the LOS is F and a breakdown has been identified. To analyze the impacts of such a breakdown, the Chapter 10 methodology must be used. No further analysis using the present chapter's methodology is possible. If demand is less than or equal to capacity, the analysis continues to Step 5.

Step 5: Estimate Speed and Density

At this point in the methodology, the following have been determined: (*a*) the FFS and appropriate FFS curve for use in the analysis and (*b*) the demand flow rate expressed in passenger cars per hour per lane under equivalent base conditions. With this information, the speed and density of the traffic stream may be estimated.

With the equations specified in Exhibit 12-6, the expected mean speed of the traffic stream can be computed. A graphical solution with Exhibit 12-7 can also be performed.

After the speed is estimated, Equation 12-11 is used to estimate the density of the traffic stream:

$$D = \frac{v_P}{S}$$

Equation 12-11

where

D = density (pc/mi/ln),

v_p = demand flow rate (pc/h/ln), and

S = mean speed of traffic stream under base conditions (mi/h).

As has been noted, Equation 12-11 is only used when v_p/c is less than or equal to 1.00. All cases in which this ratio is greater than 1.00 are LOS F. In these cases, the speed S will be outside the range of Exhibit 12-6 and Exhibit 12-7, and no speed can be estimated.

Where LOS F exists, the analyst should consult Chapter 10, which allows an analysis of the time and spatial impacts of a breakdown, including its effects on upstream and downstream segments.

Step 6: Determine LOS

Exhibit 12-15 is entered with the density obtained from Equation 12-11 to determine the expected prevailing LOS.

4. EXTENSIONS TO THE METHODOLOGY

BASIC MANAGED LANE SEGMENTS

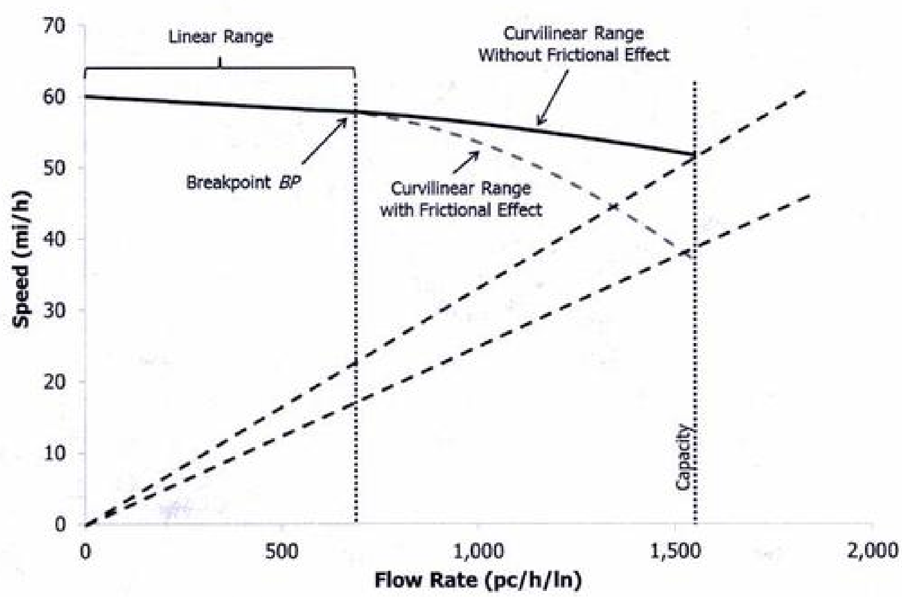
This section provides information specific to managed lanes that can be used in conjunction with the core motorized vehicle methodology to analyze the operation of basic managed lane segments on freeways. Section 2, Concepts, defines the five types of basic managed lane segments and presents basic speed–flow and capacity concepts for managed lanes.

Operating speeds and capacities of managed lanes are a function of how the managed lanes are separated from the general purpose lanes, the number of managed lanes, and, in the case of continuous access and Buffer 1 managed lane segments, operational conditions in the adjacent general purpose lanes.

The general form of the speed–flow relationship for managed lanes is illustrated in Exhibit 12-29, where the x-axis represents the adjusted 15-min demand flow rate v_p and the y-axis gives the space mean speed S_{ML} for the traffic stream.

The exhibit distinguishes two speed–flow curves that depend on a frictional effect between the managed lanes and adjacent general purpose lane. Managed lanes with continuous access or Buffer 1 separation exhibit a deteriorated performance as the general purpose lanes approach capacity (i.e., their density exceeds 35 pc/mi/ln).

Exhibit 12-29
General Form for Speed–Flow Curves for Basic Managed Lane Segments on Freeways



The general analytic form of the speed–flow relationship is given by Equation 12-12, along with the equations for determining the model parameters including the breakpoint and the capacity, both of which are based on FFS.

$$S_{ML} = \begin{cases} S_1 & v_p \leq BP \\ S_1 - S_2 - I_c \times S_3 & BP < v_p \leq c \end{cases}$$

Equation 12-12

where

S_{ML} = space mean speed of the basic managed lane segment (mi/h);

S_1 = speed within the linear portion of the speed–flow curve, from Equation 12-15 (mi/h);

S_2 = speed drop within the curvilinear portion of the speed–flow curve, from Equation 12-17 (mi/h);

S_3 = additional speed drop (mi/h) within the curvilinear portion of the speed–flow curve when the density of the adjacent general purpose lane is more than 35 pc/mi/ln, from Equation 12-19;

I_c = indicator variable, where 1 = presence of densities greater than 35 pc/mi/ln in the adjacent general purpose lane (0 or 1);

BP = breakpoint in the speed–flow curve separating the linear and curvilinear sections (pc/h/ln), from Equation 12-13; and

v_p = 15-min average flow rate (pc/h/ln).

The breakpoint in the speed–flow curve is defined by Equation 12-13:

$$BP = \left[BP_{75} + \lambda_{BP} \times \left(75 - FFS_{adj}\right)\right] \times CAF^2$$

Equation 12-13

where

BP = breakpoint in the speed–flow curve separating the linear and curvilinear sections (pc/h/ln);

BP_{75} = breakpoint for a FFS of 75 mi/h, from Exhibit 12-30 (pc/h/ln);

λ_{BP} = rate of increase in breakpoint per unit decrease in FFS, from Exhibit 12-30 (pc/h/ln);

FFS_{adj} = adjusted free-flow speed (mi/h); and

CAF = capacity adjustment factor (unitless).

Similar to general purpose lanes, capacity and FFS can be adjusted to account for the impacts of weather, incidents, and work zones and for overall calibration purposes. Research specific to managed lanes on the magnitude of these effects is limited, but the same adjustments provided for basic segments can be considered. Default CAF and SAF values for basic segments are provided in Chapter 11. The default values do not explicitly list single-lane facilities, but in the absence of field data, defaults given for two-lane facilities may be used (e.g., for a single-lane managed lane shoulder closure incident).

A basic managed lane segment's capacity is estimated by Equation 12-14:

$$c_{adj} = CAF \times \left[c_{75} - \lambda_c \times \left(75 - FFS_{adj}\right)\right]$$

Equation 12-14

where

c_{adj} = adjusted basic managed lane segment capacity (pc/h/ln);

CAF = capacity adjustment factor (unitless);

c_{75} = managed lane capacity for a FFS of 75 mi/h, from Exhibit 12-30 (pc/h/ln);

λ_c = rate of change in capacity per unit change in FFS, from Exhibit 12-30 (pc/h/ln); and

FFS_{adj} = adjusted free-flow speed (mi/h).

The linear portion of the speed–flow curve is computed from Equation 12-15:

Equation 12-15

$$S_1 = FFS_{adj} - A_1 \times \min(v_p, BP)$$

where A_1 is the speed reduction per unit of flow rate in the linear section of the speed–flow curve (mi/h), from Exhibit 12-30, and all other variables are as defined previously.

The curvilinear portion of the speed–flow curve for basic managed lane segments is characterized by using a calibration factor A_2 that is computed with Equation 12-16:

Equation 12-16

$$A_2 = A_2^{55} + \lambda_{A2} \times (FFS_{adj} - 55)$$

where

A_2 = speed reduction per unit of flow rate in the curvilinear section of the speed–flow curve (mi/h);

A_2^{55} = calibration factor for a FFS of 55 mi/h, from Exhibit 12-30 (mi/h);

λ_{A2} = rate of change in A_2 per unit increase in FFS, from Exhibit 12-30 (mi/h); and

FFS_{adj} = adjusted free-flow speed (mi/h).

The curvilinear portion of the speed–flow curve during times when the adjacent general purpose lane density is less than or equal to 35 pc/mi/ln is computed from Equation 12-17:

Equation 12-17

$$S_2 = \frac{\left(S_{1,BP} - \frac{c_{adj}}{K_c^{nf}}\right)}{(c_{adj} - BP)^{A_2}}(v_p - BP)^{A_2}$$

where

S_2 = speed drop within the curvilinear portion of the speed–flow curve (mi/h);

$S_{1,BP}$ = speed at the breakpoint of the speed–flow curve, calculated from Equation 12-15 by setting v_p to BP (mi/h);

c_{adj} = adjusted basic managed lane segment capacity (pc/h/ln);

K_c^{nf} = density at capacity, without the frictional effect of the adjacent general purpose lane, from Exhibit 12-30 (pc/mi/ln);

BP = breakpoint in the speed–flow curve separating the linear and curvilinear sections (pc/h/ln);

A_2 = speed reduction per unit of flow rate in the curvilinear section of the speed–flow curve (mi/h); and

v_p = 15-min average flow rate (pc/h/ln).

Continuous access and Buffer 1 segment types operate at lower speeds when adjacent general purpose lane density is greater than 35 pc/mi/ln. The indicator variable I_c is used to determine the status of the general purpose lane operation. This variable is determined by using Equation 12-18.

$$I_c = \begin{cases} 0 & K_{GP} \leq 35 \text{ pc/mi/ln} \\ & \text{or segment type is Buffer 2, Barrier 1, or Barrier 2} \\ 1 & \text{otherwise} \end{cases}$$

Equation 12-18

where K_{GP} is the density of the adjacent general purpose lane (pc/mi/ln).

The additional speed reduction that occurs in the curvilinear portion of the speed–flow curve because of high density in the adjacent general purpose lanes is computed by Equation 12-19:

$$S_3 = \frac{\left(\frac{c_{adj}}{K_c^{nf}}\right) - \left(\frac{c_{adj}}{K_c^f}\right)}{\left(c_{adj} - BP\right)^2} \left(v_p - BP\right)^2$$

Equation 12-19

where K_c^f is the density at capacity, with the frictional effect of the adjacent general purpose lane (pc/mi/ln), from Exhibit 12-30, and other variables are as defined previously.

Exhibit 12-30 tabulates the parameters used by speed computations for the different basic managed lane segment types.

Exhibit 12-30
Parameters for Basic Managed Lane Segment Analysis

Segment Type	BP_{75}	λ_{BP}	c_{75}	λ_c	A_2^{55}	λ_{A2}	A_1	K_c^{nf}	K_c^f
Continuous access	500	0	1,800	10	2.5	0	0	30	45
Buffer 1	600	0	1,700	10	1.4	0	0.0033	30	42[a]
Buffer 2	500	10	1,850	10	1.5	0.02	0	45[a]	NA
Barrier 1	800	0	1,750	10	1.4	0	0.004	35	NA
Barrier 2	700	20	2,100	10	1.3	0.02	0	45	NA

Note: [a] These are average values of density at capacity observed by NCHRP Project 03-96 (1), ranging from 40.9 to 42.5 pc/mi/ln for Buffer 1 and from 40.1 to 50.4 pc/mi/ln for Buffer 2 segment types.

BICYCLE METHODOLOGY FOR MULTILANE HIGHWAYS

Bicycle LOS Criteria

Bicycle levels of service for multilane highway segments are based on a bicycle LOS score, which is in turn based on a traveler perception model. Chapter 15, Two-Lane Highways, provides details about this service measure, which is identical for two-lane highways and multilane highways. The bicycle LOS score is based, in order of importance, on five variables:

Bicycle LOS is based on a traveler perception model. The measure applies only to multilane highways, not freeway segments.

- Average effective width of the outside through lane,
- Motorized vehicle volumes and speeds,
- Heavy vehicle (truck) volumes, and
- Pavement condition.

The LOS ranges for bicycles on two-lane and multilane highways are given in Exhibit 12-31.

Follow the step-by-step description of the bicycle LOS method given in Chapter 15 to calculate bicycle LOS on multilane highways.

Exhibit 12-31
Bicycle LOS for Two-Lane and Multilane Highways

LOS	Bicycle LOS Score
A	≤1.5
B	>1.5–2.5
C	>2.5–3.5
D	>3.5–4.5
E	>4.5–5.5
F	>5.5

Required Input Data

The data required for evaluating bicycle LOS on a multilane highway and the ranges of values used in the development of the LOS model (22) are as follows:

- Width of the outside through lane: 10 to 16 ft,
- Shoulder width: 0 to 6 ft,
- Motorized vehicle volumes: up to 36,000 annual average daily traffic (AADT),
- Number of directional through lanes,
- Posted speed: 25 to 50 mi/h,
- Heavy vehicle percentage: 0% to 2%, and
- Pavement condition: 2 to 5 on the FHWA 5-point pavement rating scale (23).

Methodology

The calculation of bicycle LOS on multilane and two-lane highways shares the same methodology, since multilane and two-lane highways operate in fundamentally the same manner for bicyclists and motorized vehicle drivers. Bicyclists travel much more slowly than the prevailing traffic flow and stay as far to the right as possible, and they use paved shoulders when available. This similarity indicates the need for only one model.

The bicycle LOS model for multilane highways uses a traveler perception index calibrated by using a linear regression model. The model fits independent variables associated with roadway characteristics to the results of a user survey that rates the comfort of various bicycle facilities. The resulting bicycle LOS index computes a numerical LOS score, generally ranging from 0.5 to 6.5, which is stratified to produce a LOS A to F result by using Exhibit 12-31.

Full details on the bicycle LOS methodology and calculation procedures are given in Chapter 15.

Limitations

Although the bicycle LOS model has been successfully applied to rural multilane highways, users should be aware that conditions on many of those highways are outside the range of values used to develop the model.

The bicycle methodology was developed with data collected on urban and suburban streets, including facilities that would be defined as suburban multilane highways. Although the methodology has been successfully applied to rural multilane highways in different parts of the United States, users should be aware that conditions on many rural multilane highways (i.e., posted speeds of 55 mi/h or higher or heavy vehicle percentages over 2%) will be outside the range of values used to develop the bicycle LOS model.

5. APPLICATIONS

EXAMPLE PROBLEMS

Section 6 of Chapter 26, Freeway and Highway Segments: Supplemental, provides seven example problems that go through each of the computational steps involved in applying the automobile to basic freeway and multilane highway segments:

1. Four-lane freeway LOS (operational analysis),
2. Number of lanes required to achieve a target LOS (design analysis),
3. Six-lane freeway LOS and capacity (operational and planning analysis),
4. LOS on a five-lane multilane highway with a TWLTL (operational analysis),
5. Estimation of the mixed-flow operational performance of a basic segment with a high truck percentage (operational analysis),
6. Severe weather effects on a basic freeway segment (operational analysis), and
7. Basic managed lane segment with and without friction effects (operational analysis).

Section 7 of Chapter 26 provides an example of the application of the bicycle LOS method.

RELATED CONTENT IN THE HCMAG

The *Highway Capacity Manual Applications Guide* (HCMAG), accessible through the online HCM Volume 4, provides guidance on applying the HCM on basic freeway segments. Case Study 4 goes through the process of identifying the goals, objectives, and analysis tools for investigating LOS on a 3-mi section of New York State Route 7 in Albany. The case study applies the analysis tools to assess the performance of the route, to identify areas that are deficient, and to investigate alternatives for correcting the deficiencies.

This case study includes the following problems related to basic freeway segments:

1. Problem 1: Analysis of two basic freeway segments
   a. Subproblem 1a: Traffic flow patterns
   b. Subproblem 1b: Selection of appropriate data and basic freeway analysis
   c. Subproblem 1c: Basic freeway analysis
2. Problem 4: Analysis of segments as part of an extended freeway facility
   a. Subproblem 4a: Separation of Route 7 for HCM analysis
   b. Subproblem 4b: Study of off-peak periods
   c. Subproblem 4c: What is the operational performance of Route 7 during the peak period?

Although the HCMAG was based on the HCM2000's procedures and chapter organization, the general thought process described in its case studies is applicable to this edition of the HCM.

EXAMPLE RESULTS

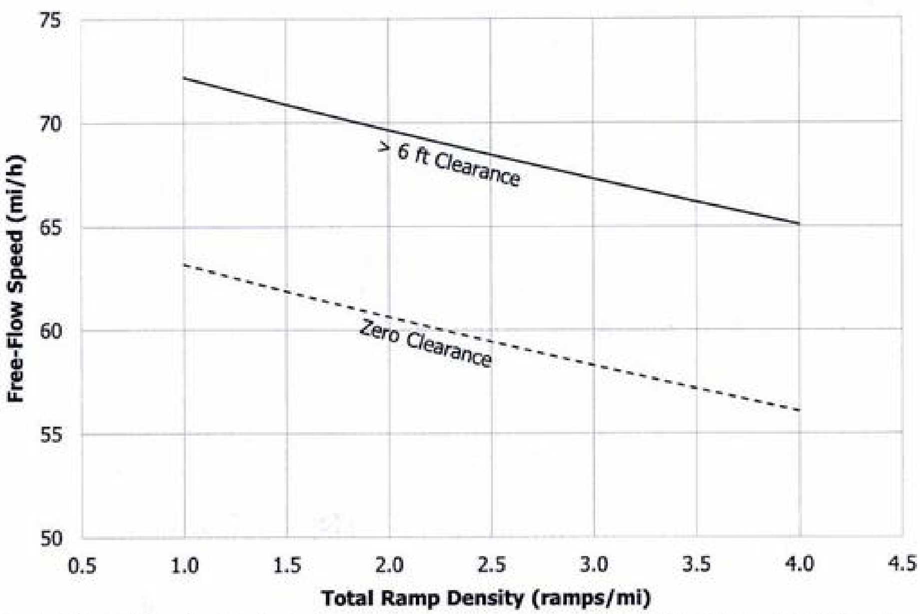
This section presents the results of applying this chapter's method in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs, as well as to help evaluate whether their analysis results are reasonable. The exhibits in this section are not intended to substitute for an analysis and are deliberately provided in a format large enough to depict general trends in the results but not large enough to pull out specific results.

Sensitivity of Freeway Results to Total Ramp Density and Right-Side Lateral Clearance

The freeway FFS is most sensitive to the total ramp density and the right-side lateral clearance.

Exhibit 12-32 illustrates how FFS varies for a basic freeway segment with a base FFS of 75 mi/h when the total ramp density varies from 1 to 4 ramps/mi. The top curve shows the case with adequate right-side clearance (i.e., 6 ft or greater), while the lower curve shows the case with no right-side clearance (i.e., no right shoulder).

Exhibit 12-32
Illustrative Effect of Total Ramp Density and Right-Side Lateral Clearance on Basic Freeway Segment FFS



Note: Calculated by using this chapter's methods. Fixed values include BFFS = 75.4 mi/h for a basic freeway segment and f_{LW} = 6.6 for 10-ft lanes.

Each on- and off-ramp in the direction of travel is counted when total ramp density is determined.

A freeway with 2 ramps/mi represents a case where there are 6 ramps within 3 mi on either side of the study location. This occurs primarily in urban areas, where interchanges may be close to each other, sometimes even in excess of 6 ramps/mi. The FFS for that condition is nearly 70 mi/h, assuming a base FFS of 75 mi/h. In contrast, the same segment without any right-side clearance has a much lower FFS—just above 60 mi/h.

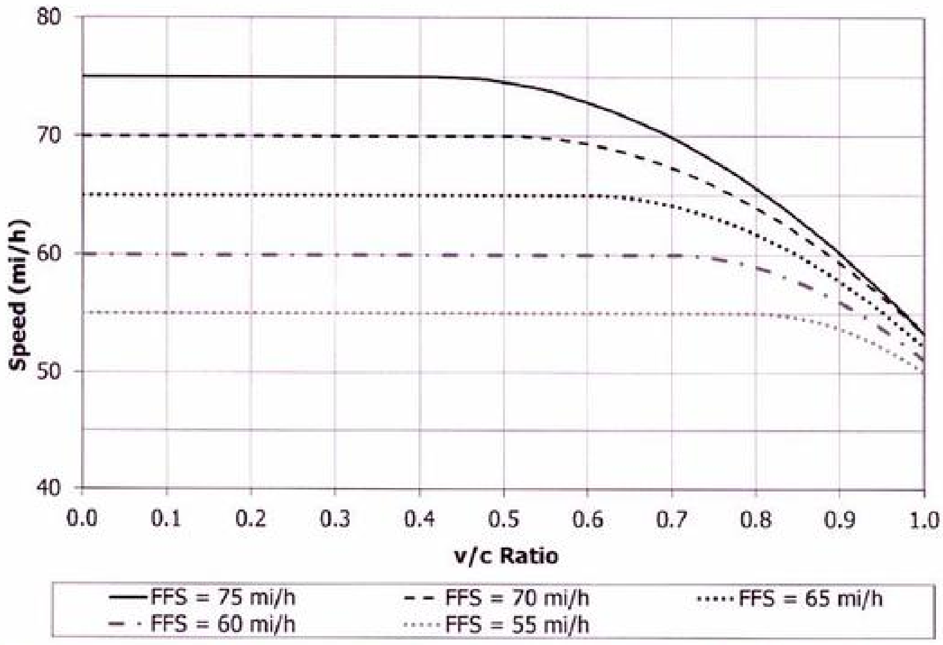
In general, most interchanges involve two to four ramps. A full cloverleaf, for example, has four ramps: two on-ramps and two off-ramps in each direction. A diamond interchange has two ramps in each direction: one on-ramp and one off-ramp. Thus, a freeway with two cloverleaf interchanges fully contained within 1 mi would have a total ramp density of 8 ramps/mi. A freeway with two diamond interchanges fully contained within 1 mi would have a total density of 4 ramps/mi. This suggests that in any given situation (with comparable demand flows), cloverleaf interchanges will have a greater negative impact on FFS than diamond interchanges.

Although the curves in Exhibit 12-32 are not straight lines, their slopes are relatively constant. On average, an increase of 2 ramps/mi in total ramp density causes a drop in FFS of approximately 5 mi/h. A reduction in FFS, of course, implies reductions in capacity and service volumes.

Sensitivity of Freeway Results to *v/c* Ratio

Exhibit 12-33 shows the relationship between speed and *v/c* ratio. Not unexpectedly, the shapes of these curves are similar to the basic speed–flow curves of Exhibit 12-7. Speed does not begin to decline until a *v/c* ratio of 0.42 to 0.80 is reached, depending on the FFS.

Exhibit 12-33
Illustrative Effect of *v/c* Ratio on Basic Freeway Segment Speed

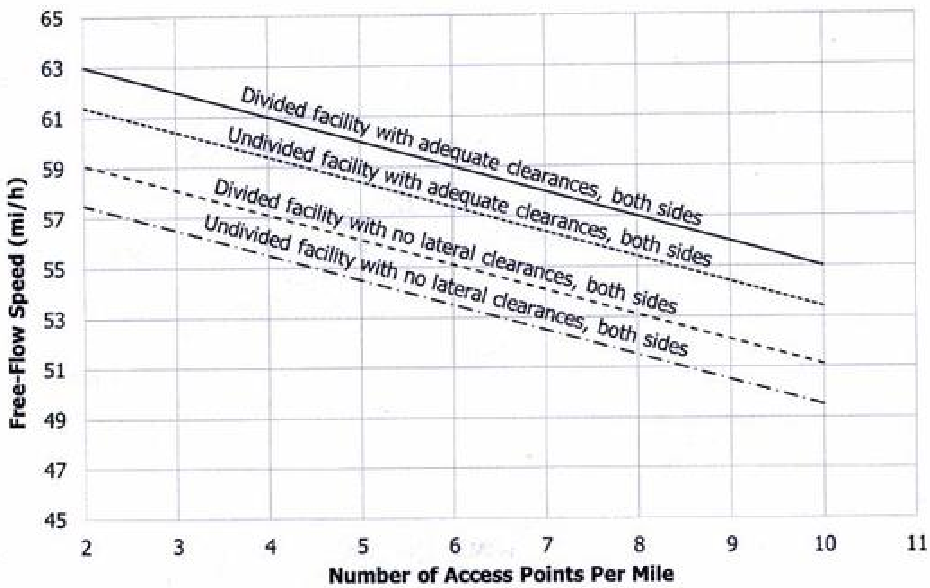


Note: Calculated by using this chapter's methods. Fixed values include CAF = 1.0, SAF = 1.0, and no heavy vehicle or grade effects.

Sensitivity of Multilane Highway Results to Access Point Density, Lateral Clearance, and Median Type

Exhibit 12-34 illustrates the effect of access point density, lateral clearance, and median type (divided or undivided) on the resulting multilane highway segment FFS, assuming a base FFS of 65 mi/h.

Exhibit 12-34
Illustrative Effect of Access Point Density, Lateral Clearance, and Median Type on Multilane Highway Segment FFS



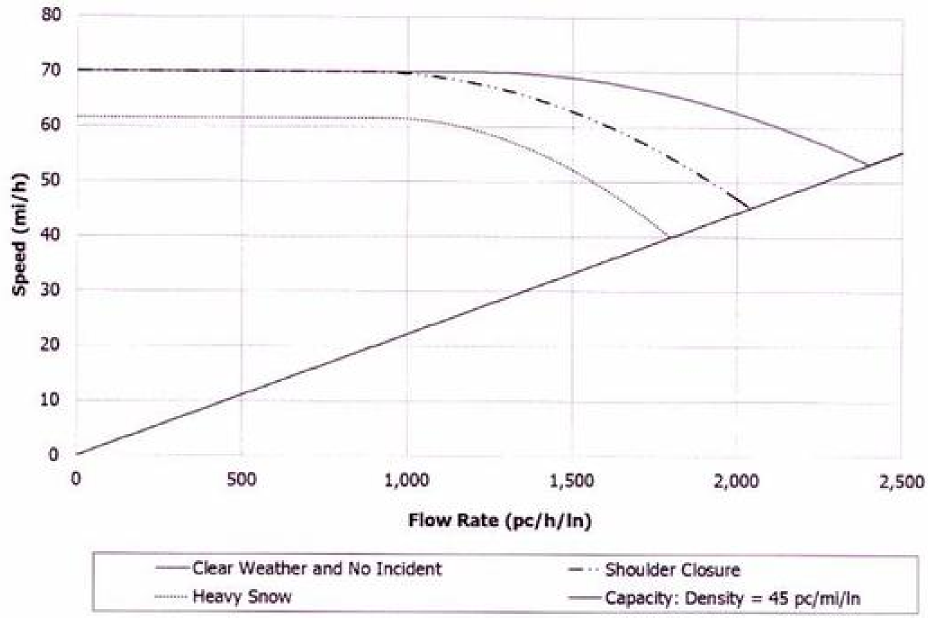
Note: Calculated by using this chapter's methods. Fixed values include base FFS = 65 mi/h and f_{LW} = 0 for 12-ft lanes.

Exhibit 12-34 shows that adding a single access point per mile results in a 1-mi/h drop in the FFS. This value represents the slope of all four lines in the exhibit. The effect of lateral clearance is also significant; the FFS is reduced by nearly 4 mi/h when all other parameters are held fixed. Finally, the FFS of a divided segment is 1.6 mi/h higher than that of an undivided segment when clearances and the number of access points are both controlled for.

Sensitivity of Freeway Results to Incidents and Inclement Weather

The speed–flow curves presented in this chapter are primarily sensitive to flow rates, FFS, and capacity. Incidents and inclement weather reduce a basic freeway segment's capacity and therefore indirectly reduce its FFS. Inclement weather also produces a direct reduction in FFS. Exhibit 12-35 shows speed–flow curves for a basic freeway segment for three different conditions—base condition, shoulder-closure incident, and heavy snow—for a base FFS of 70 mi/h. The CAFs used for shoulder closure and heavy snow are 0.85 and 0.776, respectively, on the basis of default values from Chapter 11, while the SAF for heavy snow is 0.88.

Exhibit 12-35
Illustrative Effect of Incidents and Inclement Weather on Basic Freeway Segment FFS

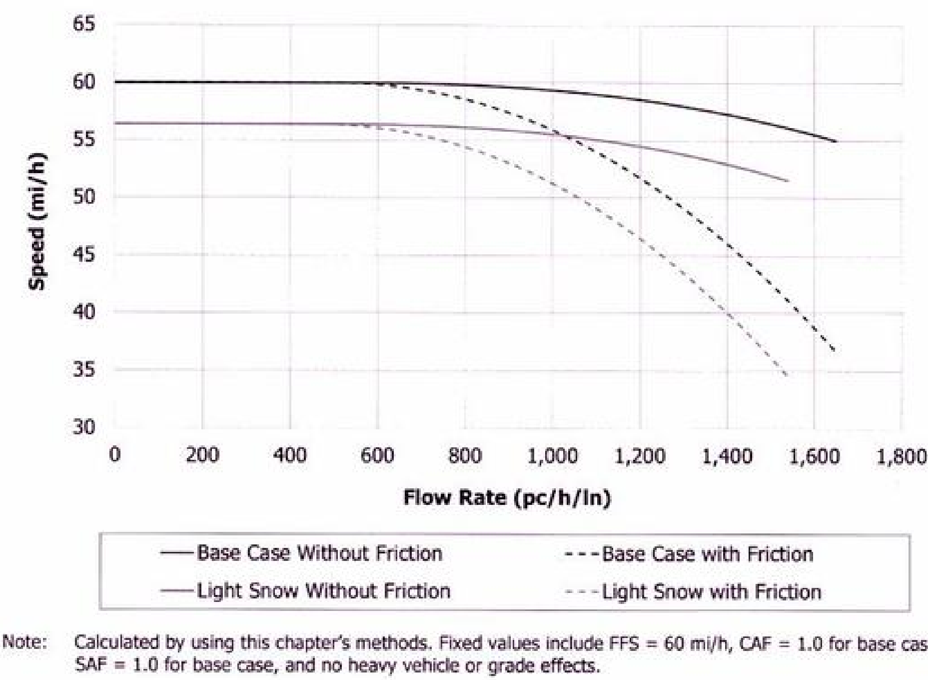


Note: Calculated by using this chapter's methods. Fixed values include FFS = 70 mi/h, CAF = 1.0 for base case, SAF = 1.0 for base case, and no heavy vehicle or grade effects.

Sensitivity of Managed Lane Results to Inclement Weather and General Purpose Lane Friction

Exhibit 12-36 depicts speed–flow curves for a single-lane continuous access managed lane segment for combinations of weather (light snow and nonsevere) and adjacent general purpose lane density (≤35 pc/mi/ln, resulting in no friction, and >35 pc/mi/ln, resulting in friction). The CAF for light snow is 0.957 and the SAF for light snow is 0.94, on the basis of default values from Chapter 11.

Exhibit 12-36
Illustrative Effect of Inclement Weather and General Purpose Lane Friction on Managed Lane FFS



Note: Calculated by using this chapter's methods. Fixed values include FFS = 60 mi/h, CAF = 1.0 for base case, SAF = 1.0 for base case, and no heavy vehicle or grade effects.

PLANNING AND PRELIMINARY ENGINEERING ANALYSIS

Planning and preliminary engineering applications also find the number of lanes required to deliver a target LOS but provide more generalized input values to the methodology.

A frequent objective of planning or preliminary engineering analysis is to develop a general idea of the number of lanes that will be required to deliver a target LOS. The primary differences are that many default values will be used and the demand volume will usually be expressed as an AADT. Thus, a planning and preliminary engineering analysis starts by converting the demand expressed as an AADT to an estimate of the directional peak-hour demand volume (DDHV) with Equation 12-20:

Equation 12-20

$$V = DDHV = AADT \times K \times D$$

where K is the proportion of AADT occurring during the peak hour and D is the proportion of peak-hour volume traveling in the peak direction.

Chapter 3 provides additional guidance on K- and D-factors.

On urban freeways, the typical range of K-factors is from 0.08 to 0.10. On rural freeways, values typically range between 0.09 and 0.13. Directional distributions also vary, as illustrated in Chapter 3, Modal Characteristics, but a typical value for both urban and rural freeways is 0.55. As with all default values, locally or regionally calibrated values are preferred and yield more accurate results. Both the K-factor and the D-factor have a significant impact on the estimated hourly demand volume.

Once the hourly demand volume is estimated, the methodology follows the same path as that for design analysis, described next. Additional details and discussion on planning applications can be found in the *Planning and Preliminary Engineering Applications Guide to the HCM* in Volume 4.

DESIGN ANALYSIS

Design analyses find the number of lanes required for a target LOS, given a specified demand volume.

In design analysis, a known demand volume is used to determine the number of lanes needed to deliver a target LOS. Two modifications are required to the operational analysis methodology. First, since the number of lanes is to be determined, the demand volume is converted to a demand flow rate in passenger cars per hour, not per lane, by using Equation 12-21 instead of Equation 12-9:

Equation 12-21

$$v = \frac{V}{PHF \times f_{HV}}$$

where v is the demand flow rate in passenger cars per hour and all other variables are as previously defined.

Second, a maximum service flow rate for the target LOS is then selected from Exhibit 12-37 for basic freeway segments or Exhibit 12-38 for multilane highways. These values are selected from the base speed–flow curves of Exhibit 12-6 for each LOS. In using these exhibits, the FFS should be rounded to the nearest 5 mi/h, and no interpolation is permitted.

Exhibit 12-37
Maximum Service Flow Rates for Basic Freeway Segments Under Base Conditions

FFS (mi/h)	Maximum Service Flow Rates for Target LOS (pc/h/ln) A	B	C	D	E
75	820	1,310	1,750	2,110	2,400
70	770	1,250	1,690	2,080	2,400
65	710	1,170	1,630	2,030	2,350
60	660	1,080	1,560	2,010	2,300
55	600	990	1,430	1,900	2,250

Note: All values rounded to the nearest 10 pc/h/ln.

FFS (mi/h)	Maximum Service Flow Rates for Target LOS (pc/h/ln) A	B	C	D	E
60	660	1,080	1,550	1,980	2,200
55	600	990	1,430	1,850	2,100
50	550	900	1,300	1,710	2,000
45	290	810	1,170	1,550	1,900

Exhibit 12-38
Maximum Service Flow Rates for Multilane Highway Segments Under Base Conditions

Next, the number of lanes required to deliver the target LOS can be found from Equation 12-22:

$$N = \frac{v}{MSF_i}$$

Equation 12-22

where N is the number of lanes required (ln) and MSF_i is the maximum service flow rate for LOS i (pc/h/ln) from Exhibit 12-37 or Exhibit 12-38.

Equation 12-21 and Equation 12-22 can be conveniently combined as Equation 12-23:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV}}$$

Equation 12-23

where all variables are as previously defined.

All fractional values of N must be rounded up.

The value of N resulting from Equation 12-22 or Equation 12-23 will most likely be fractional. Since only integer numbers of lanes can be constructed, the result is always rounded to the next-higher value. Thus, if the result is 3.2 lanes, 4 must be provided. The 3.2 lanes is, in effect, the minimum number of lanes needed to provide the target LOS. If the result were rounded to 3, a poorer LOS than the target value would result.

Because only whole lanes can be built, the target LOS for a given demand volume may not be achievable.

The rounding-up process will occasionally produce an interesting result: a target LOS (for example, LOS C) may not be achievable for a given demand volume. If 2.1 lanes are required to produce LOS C, providing 2 lanes would drop the LOS, most likely to D. However, if three lanes are provided, the LOS might improve to B. Some judgment may be required to interpret the results. In this case, two lanes might be provided even though they would result in a borderline LOS D. Economic considerations might lead a decision maker to accept a lower operating condition than that originally targeted.

SERVICE FLOW RATES, SERVICE VOLUMES, AND DAILY SERVICE VOLUMES

This chapter's methodology can be easily manipulated to produce service flow rates, service volumes, and daily service volumes for basic freeway segments and multilane highways.

Exhibit 12-37 gave values of the maximum hourly service flow rates MSF_i for each LOS for freeways of varying FFS. These values are given in terms of passenger cars per hour per lane under equivalent base conditions. A service flow rate SF_i is the maximum rate of flow that can exist while LOS i is maintained during the 15-min analysis period under prevailing conditions. It can be computed from the maximum service flow rate by using Equation 12-24:

$$SF_i = MSF_i \times N \times f_{HV}$$

Equation 12-24

where all variables are as previously defined.

A service flow rate can be converted to a service volume SV_i by applying a PHF, as shown in Equation 12-25. A service volume is the maximum hourly volume that can exist while LOS i is maintained during the worst 15-min period of the analysis hour.

Equation 12-25

$$SV_i = SF_i \times PHF$$

where all variables are as previously defined.

A daily service volume DSV_i is the maximum AADT that can be accommodated by the facility under prevailing conditions while LOS i is maintained during the worst 15-min period of the analysis day. It is estimated from Equation 12-26:

Equation 12-26

$$DSV_i = \frac{SV_i}{K \times D} = \frac{MSF_i \times N \times f_{HV} \times PHF}{K \times D}$$

where all variables are as previously defined.

Service flow rates *SF* and service volumes *SV* are stated for a single direction. Daily service volumes *DSV* are stated as total volumes in *both* directions of the freeway or multilane highway.

This method can also be used to develop daily service volume tables for basic managed lane segments by using regional assumptions about the various input parameters.

Generalized Daily Service Volumes for Basic Freeway Segments

Exhibit 12-39 and Exhibit 12-40 show generalized daily service volume tables for urban and rural basic freeway segments, respectively. They are based on the following set of typical conditions:

- Percent heavy vehicles = 5% (urban), 12% (rural);
- FFS = 70 mi/h; and
- PHF = 0.94.

Values of rural and urban daily service volumes are provided for four-lane, six-lane, and eight-lane freeways in level and rolling terrain. A range of *K*- and *D*-factors is provided. Users should enter Exhibit 12-39 and Exhibit 12-40 with local or regional values of these factors for the appropriate size of freeway in the appropriate terrain.

Exhibit 12-39
Daily Service Volume Table for Urban Basic Freeway Segments (1,000 veh/day)

		Four-Lane Freeways				Six-Lane Freeways				Eight-Lane Freeways			
K	*D*	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
						Level Terrain							
0.08	0.50	56.4	77.6	94.6	107.4	84.6	116.4	141.8	161.1	112.7	155.2	189.1	214.9
	0.55	51.2	70.6	86.0	97.7	76.9	105.9	129.0	146.5	102.5	141.1	171.9	195.3
	0.60	47.0	64.7	78.8	89.5	70.5	97.0	118.2	134.3	93.9	129.4	157.6	179.0
	0.65	43.4	59.7	72.7	82.6	65.0	89.6	109.1	124.0	86.7	119.4	145.5	165.3
0.09	0.50	50.1	69.0	84.1	95.5	75.2	103.5	126.1	143.2	100.2	138.0	168.1	191.0
	0.55	45.5	62.7	76.4	86.8	68.3	94.1	114.6	130.2	91.1	125.5	152.8	173.6
	0.60	41.8	57.5	70.0	79.6	62.6	86.2	105.1	119.4	83.5	115.0	140.1	159.2
	0.65	38.5	53.1	64.7	73.5	57.8	79.6	97.0	110.2	77.1	106.2	129.3	146.9
0.10	0.50	45.1	62.1	75.7	85.9	67.6	93.1	113.5	128.9	90.2	124.2	151.3	171.9
	0.55	41.0	56.5	68.8	78.1	61.5	84.7	103.2	117.2	82.0	112.9	137.5	156.3
	0.60	37.6	51.7	63.0	71.6	56.4	77.6	94.6	107.4	75.2	103.5	126.1	143.2
	0.65	34.7	47.8	58.2	66.1	52.0	71.7	87.3	99.2	69.4	95.5	116.4	132.2
0.11	0.50	41.0	56.5	68.8	78.1	61.5	84.7	103.2	117.2	82.0	112.9	137.5	156.3
	0.55	37.3	51.3	62.5	71.0	55.9	77.0	93.8	106.5	74.5	102.6	125.0	142.1
	0.60	34.2	47.0	57.3	65.1	51.2	70.6	86.0	97.7	68.3	94.1	114.6	130.2
	0.65	31.5	43.4	52.9	60.1	47.3	65.1	79.4	90.1	63.1	86.9	105.8	120.2
0.12	0.50	37.6	51.7	63.0	71.6	56.4	77.6	94.6	107.4	75.2	103.5	126.1	143.2
	0.55	34.2	47.0	57.3	65.1	51.2	70.6	86.0	97.7	68.3	94.1	114.6	130.2
	0.60	31.3	43.1	52.5	59.7	47.0	64.7	78.8	89.5	62.6	86.2	105.1	119.4
	0.65	28.9	39.8	48.5	55.1	43.4	59.7	72.7	82.6	57.8	79.6	97.0	110.2
						Rolling Terrain							
0.08	0.50	53.8	74.1	90.3	102.5	80.7	111.1	135.4	153.8	107.6	148.2	180.5	205.1
	0.55	48.9	67.4	82.1	93.2	73.4	101.0	123.1	139.8	97.8	134.7	164.1	186.4
	0.60	44.8	61.7	75.2	85.5	67.3	92.6	112.8	128.2	89.7	123.5	150.4	170.9
	0.65	41.4	57.0	69.4	78.9	62.1	85.5	104.2	118.3	82.8	114.0	138.9	157.8
0.09	0.50	47.8	65.9	80.2	91.2	71.7	98.8	120.4	136.7	95.7	131.7	160.5	182.3
	0.55	43.5	59.9	72.9	82.9	65.2	89.8	109.4	124.3	87.0	119.7	145.9	165.7
	0.60	39.9	54.9	66.9	76.0	59.8	82.3	100.3	113.9	79.7	109.8	133.7	151.9
	0.65	36.8	50.7	61.7	70.1	55.2	76.0	92.6	105.2	73.6	101.3	123.4	140.2
0.10	0.50	43.0	59.3	72.2	82.0	64.6	88.9	108.3	123.1	86.1	118.6	144.4	164.1
	0.55	39.1	53.9	65.6	74.6	58.7	80.8	98.5	111.9	78.3	107.8	131.3	149.2
	0.60	35.9	49.4	60.2	68.4	53.8	74.1	90.3	102.5	71.7	98.8	120.4	136.7
	0.65	33.1	45.6	55.5	63.1	49.7	68.4	83.3	94.7	66.2	91.2	111.1	126.2
0.11	0.50	39.1	53.9	65.6	74.6	58.7	80.8	98.5	111.9	78.3	107.8	131.3	149.2
	0.55	35.6	49.0	59.7	67.8	53.4	73.5	89.5	101.7	71.1	98.0	119.4	135.6
	0.60	32.6	44.9	54.7	62.1	48.9	67.4	82.1	93.2	65.2	89.8	109.4	124.3
	0.65	30.1	41.5	50.5	57.4	45.2	62.2	75.7	86.1	60.2	82.9	101.0	114.7
0.12	0.50	35.9	49.4	60.2	68.4	53.8	74.1	90.3	102.5	71.7	98.8	120.4	136.7
	0.55	32.6	44.9	54.7	62.1	48.9	67.4	82.1	93.2	65.2	89.8	109.4	124.3
	0.60	29.9	41.2	50.1	57.0	44.8	61.7	75.2	85.5	59.8	82.3	100.3	113.9
	0.65	27.6	38.0	46.3	52.6	41.4	57.0	69.4	78.9	55.2	76.0	92.6	105.2

Note: Key assumptions: 5% trucks, PHF = 0.94, FFS = 70 mi/h.

Exhibit 12-40
Daily Service Volume Table for Rural Basic Freeway Segments (1,000 veh/day)

		Four-Lane Freeways				Six-Lane Freeways				Eight-Lane Freeways			
K	*D*	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
						Level Terrain							
0.08	0.50	52.8	72.8	88.7	100.7	79.3	109.2	133.0	151.1	105.7	145.5	177.3	201.4
	0.55	48.0	66.2	80.6	91.6	72.1	99.2	120.9	137.3	96.1	132.3	161.2	183.1
	0.60	44.0	60.6	73.9	83.9	66.1	91.0	110.8	125.9	88.1	121.3	147.8	167.9
	0.65	40.6	56.0	68.2	77.5	61.0	84.0	102.3	116.2	81.3	112.0	136.4	154.9
0.09	0.50	47.0	64.7	78.8	89.5	70.5	97.0	118.2	134.3	93.9	129.4	157.6	179.0
	0.55	42.7	58.8	71.6	81.4	64.1	88.2	107.5	122.1	85.4	117.6	143.3	162.8
	0.60	39.1	53.9	65.7	74.6	58.7	80.9	98.5	111.9	78.3	107.8	131.3	149.2
	0.65	36.1	49.8	60.6	68.9	54.2	74.6	90.9	103.3	72.3	99.5	121.2	137.7
0.10	0.50	42.3	58.2	70.9	80.6	63.4	87.3	106.4	120.9	84.6	116.4	141.8	161.1
	0.55	38.4	52.9	64.5	73.2	57.6	79.4	96.7	109.9	76.9	105.9	129.0	146.5
	0.60	35.2	48.5	59.1	67.1	52.8	72.8	88.7	100.7	70.5	97.0	118.2	134.3
	0.65	32.5	44.8	54.6	62.0	48.8	67.2	81.8	93.0	65.0	89.6	109.1	124.0
0.11	0.50	38.4	52.9	64.5	73.2	57.6	79.4	96.7	109.9	76.9	105.9	129.0	146.5
	0.55	34.9	48.1	58.6	66.6	52.4	72.2	87.9	99.9	69.9	96.2	117.2	133.2
	0.60	32.0	44.1	53.7	61.0	48.0	66.2	80.6	91.6	64.1	88.2	107.5	122.1
	0.65	29.6	40.7	49.6	56.3	44.3	61.1	74.4	84.5	59.1	81.4	99.2	112.7
0.12	0.50	35.2	48.5	59.1	67.1	52.8	72.8	88.7	100.7	70.5	97.0	118.2	134.3
	0.55	32.0	44.1	53.7	61.0	48.0	66.2	80.6	91.6	64.1	88.2	107.5	122.1
	0.60	29.4	40.4	49.3	56.0	44.0	60.6	73.9	83.9	58.7	80.9	98.5	111.9
	0.65	27.1	37.3	45.5	51.6	40.6	56.0	68.2	77.5	54.2	74.6	90.9	103.3
						Rolling Terrain							
0.08	0.50	47.7	65.7	80.1	91.0	71.6	98.6	120.1	136.5	95.5	131.5	160.1	181.9
	0.55	43.4	59.8	72.8	82.7	65.1	89.6	109.2	124.0	86.8	119.5	145.6	165.4
	0.60	39.8	54.8	66.7	75.8	59.7	82.2	100.1	113.7	79.6	109.5	133.5	151.6
	0.65	36.7	50.6	61.6	70.0	55.1	75.8	92.4	105.0	73.4	101.1	123.2	140.0
0.09	0.50	42.4	58.4	71.2	80.9	63.6	87.6	106.8	121.3	84.9	116.9	142.4	161.7
	0.55	38.6	53.1	64.7	73.5	57.9	79.7	97.1	110.3	77.1	106.2	129.4	147.0
	0.60	35.4	48.7	59.3	67.4	53.0	73.0	89.0	101.1	70.7	97.4	118.6	134.8
	0.65	32.6	44.9	54.8	62.2	49.0	67.4	82.1	93.3	65.3	89.9	109.5	124.4
0.10	0.50	38.2	52.6	64.1	72.8	57.3	78.9	96.1	109.2	76.4	105.2	128.1	145.5
	0.55	34.7	47.8	58.2	66.2	52.1	71.7	87.4	99.2	69.4	95.6	116.5	132.3
	0.60	31.8	43.8	53.4	60.6	47.7	65.7	80.1	91.0	63.6	87.6	106.8	121.3
	0.65	29.4	40.4	49.3	56.0	44.1	60.7	73.9	84.0	58.7	80.9	98.6	112.0
0.11	0.50	34.7	47.8	58.2	66.2	52.1	71.7	87.4	99.2	69.4	95.6	116.5	132.3
	0.55	31.6	43.5	52.9	60.1	47.3	65.2	79.4	90.2	63.1	86.9	105.9	120.3
	0.60	28.9	39.8	48.5	55.1	43.4	59.8	72.8	82.7	57.9	79.7	97.1	110.3
	0.65	26.7	36.8	44.8	50.9	40.1	55.2	67.2	76.3	53.4	73.5	89.6	101.8
0.12	0.50	31.8	43.8	53.4	60.6	47.7	65.7	80.1	91.0	63.6	87.6	106.8	121.3
	0.55	28.9	39.8	48.5	55.1	43.4	59.8	72.8	82.7	57.9	79.7	97.1	110.3
	0.60	26.5	36.5	44.5	50.5	39.8	54.8	66.7	75.8	53.0	73.0	89.0	101.1
	0.65	24.5	33.7	41.1	46.7	36.7	50.6	61.6	70.0	49.0	67.4	82.1	93.3

Note: Key assumptions: 12% trucks, PHF = 0.94, FFS = 70 mi/h.

Generalized Daily Service Volumes for Multilane Highways

Exhibit 12-41 and Exhibit 12-42 are generalized daily service volume tables for urban and rural multilane highways, respectively. They are based on the following set of typical conditions:

- Percent heavy vehicles = 8% (urban), 12% (rural);
- FFS = 60 mi/h; and
- PHF = 0.95 (urban), 0.88 (rural).

Daily service volumes are provided for four-, six-, and eight-lane highways in level and rolling terrain. A range of *K*- and *D*-factors is provided. Users should enter Exhibit 12-41 and Exhibit 12-42 with local or regional values of these factors for the appropriate size of multilane highway in the appropriate terrain.

Exhibit 12-41
Generalized Daily Service Volumes for Urban Multilane Highways (1,000 veh/day)

		Four-Lane Highways				Six-Lane Highways				Eight-Lane Highways			
K	*D*	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
						Level Terrain							
0.08	0.50	47.5	68.2	84.9	96.8	71.3	102.2	127.3	145.1	95.0	136.3	169.7	193.5
	0.55	43.2	62.0	77.2	88.0	64.8	93.0	115.7	131.9	86.4	123.9	154.3	175.9
	0.60	39.6	56.8	70.7	80.6	59.4	85.2	106.1	120.9	79.2	113.6	141.4	161.3
	0.65	36.5	52.4	65.3	74.4	54.8	78.7	97.9	111.6	73.1	104.9	130.6	148.9
0.09	0.50	42.2	60.6	75.4	86.0	63.3	90.9	113.2	129.0	84.4	121.2	150.9	172.0
	0.55	38.4	55.1	68.6	78.2	57.6	82.6	102.9	117.3	76.8	110.2	137.2	156.4
	0.60	35.2	50.5	62.9	71.7	52.8	75.7	94.3	107.5	70.4	101.0	125.7	143.3
	0.65	32.5	46.6	58.0	66.2	48.7	69.9	87.0	99.2	65.0	93.2	116.1	132.3
0.10	0.50	38.0	54.5	67.9	77.4	57.0	81.8	101.8	116.1	76.0	109.1	135.8	154.8
	0.55	34.5	49.6	61.7	70.4	51.8	74.4	92.6	105.6	69.1	99.1	123.4	140.7
	0.60	31.7	45.4	56.6	64.5	47.5	68.2	84.9	96.8	63.3	90.9	113.2	129.0
	0.65	29.2	41.9	52.2	59.5	43.8	62.9	78.3	89.3	58.5	83.9	104.5	119.1
0.11	0.50	34.5	49.6	61.7	70.4	51.8	74.4	92.6	105.6	69.1	99.1	123.4	140.7
	0.55	31.4	45.1	56.1	64.0	47.1	67.6	84.2	96.0	62.8	90.1	112.2	127.9
	0.60	28.8	41.3	51.4	58.6	43.2	62.0	77.2	88.0	57.6	82.6	102.9	117.3
	0.65	26.6	38.1	47.5	54.1	39.9	57.2	71.2	81.2	53.1	76.3	95.0	108.3
0.12	0.50	31.7	45.4	56.6	64.5	47.5	68.2	84.9	96.8	63.3	90.9	113.2	129.0
	0.55	28.8	41.3	51.4	58.6	43.2	62.0	77.2	88.0	57.6	82.6	102.9	117.3
	0.60	26.4	37.9	47.1	53.8	39.6	56.8	70.7	80.6	52.8	75.7	94.3	107.5
	0.65	24.4	35.0	43.5	49.6	36.5	52.4	65.3	74.4	48.7	69.9	87.0	99.2
						Rolling Terrain							
0.08	0.50	44.2	63.5	79.0	90.1	66.3	95.2	118.5	135.1	88.4	126.9	158.0	180.2
	0.55	40.2	57.7	71.8	81.9	60.3	86.5	107.7	122.8	80.4	115.4	143.7	163.8
	0.60	36.9	52.9	65.8	75.1	55.3	79.3	98.8	112.6	73.7	105.8	131.7	150.1
	0.65	34.0	48.8	60.8	69.3	51.0	73.2	91.2	103.9	68.0	97.6	121.6	138.6
0.09	0.50	39.3	56.4	70.2	80.1	59.0	84.6	105.4	120.1	78.6	112.8	140.5	160.2
	0.55	35.7	51.3	63.8	72.8	53.6	76.9	95.8	109.2	71.5	102.6	127.7	145.6
	0.60	32.8	47.0	58.5	66.7	49.1	70.5	87.8	100.1	65.5	94.0	117.1	133.5
	0.65	30.2	43.4	54.0	61.6	45.4	65.1	81.0	92.4	60.5	86.8	108.1	123.2
0.10	0.50	35.4	50.8	63.2	72.1	53.1	76.2	94.8	108.1	70.8	101.5	126.4	144.1
	0.55	32.2	46.2	57.5	65.5	48.2	69.2	86.2	98.3	64.3	92.3	114.9	131.0
	0.60	29.5	42.3	52.7	60.1	44.2	63.5	79.0	90.1	59.0	84.6	105.4	120.1
	0.65	27.2	39.1	48.6	55.4	40.8	58.6	72.9	83.2	54.4	78.1	97.2	110.9
0.11	0.50	32.2	46.2	57.5	65.5	48.2	69.2	86.2	98.3	64.3	92.3	114.9	131.0
	0.55	29.2	42.0	52.2	59.6	43.9	62.9	78.4	89.3	58.5	83.9	104.5	119.1
	0.60	26.8	38.5	47.9	54.6	40.2	57.7	71.8	81.9	53.6	76.9	95.8	109.2
	0.65	24.7	35.5	44.2	50.4	37.1	53.3	66.3	75.6	49.5	71.0	88.4	100.8
0.12	0.50	29.5	42.3	52.7	60.1	44.2	63.5	79.0	90.1	59.0	84.6	105.4	120.1
	0.55	26.8	38.5	47.9	54.6	40.2	57.7	71.8	81.9	53.6	76.9	95.8	109.2
	0.60	24.6	35.3	43.9	50.0	36.9	52.9	65.8	75.1	49.1	70.5	87.8	100.1
	0.65	22.7	32.5	40.5	46.2	34.0	48.8	60.8	69.3	45.4	65.1	81.0	92.4

Note: Key assumptions: 8% trucks, PHF = 0.95, FFS = 60 mi/h.

Exhibit 12-42
Generalized Daily Service Volumes for Rural Multilane Highways (1,000 veh/day)

		Four-Lane Highways				Six-Lane Highways				Eight-Lane Highways			
K	*D*	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
						Level Terrain							
0.08	0.50	42.4	60.9	75.8	86.4	63.6	91.3	113.7	129.6	84.9	121.8	151.6	172.9
	0.55	38.6	55.4	68.9	78.6	57.9	83.0	103.4	117.9	77.1	110.7	137.8	157.1
	0.60	35.4	50.7	63.2	72.0	53.0	76.1	94.8	108.0	70.7	101.5	126.3	144.0
	0.65	32.6	46.8	58.3	66.5	49.0	70.3	87.5	99.7	65.3	93.7	116.6	133.0
0.09	0.50	37.7	54.1	67.4	76.8	56.6	81.2	101.1	115.2	75.4	108.2	134.8	153.7
	0.55	34.3	49.2	61.3	69.8	51.4	73.8	91.9	104.8	68.6	98.4	122.5	139.7
	0.60	31.4	45.1	56.2	64.0	47.1	67.7	84.2	96.0	62.9	90.2	112.3	128.0
	0.65	29.0	41.6	51.8	59.1	43.5	62.4	77.7	88.6	58.0	83.3	103.7	118.2
0.10	0.50	33.9	48.7	60.6	69.1	50.9	73.1	91.0	103.7	67.9	97.4	121.3	138.3
	0.55	30.9	44.3	55.1	62.9	46.3	66.4	82.7	94.3	61.7	88.6	110.3	125.7
	0.60	28.3	40.6	50.5	57.6	42.4	60.9	75.8	86.4	56.6	81.2	101.1	115.2
	0.65	26.1	37.5	46.6	53.2	39.2	56.2	70.0	79.8	52.2	74.9	93.3	106.4
0.11	0.50	30.9	44.3	55.1	62.9	46.3	66.4	82.7	94.3	61.7	88.6	110.3	125.7
	0.55	28.1	40.3	50.1	57.1	42.1	60.4	75.2	85.7	56.1	80.5	100.2	114.3
	0.60	25.7	36.9	45.9	52.4	38.6	55.4	68.9	78.6	51.4	73.8	91.9	104.8
	0.65	23.7	34.1	42.4	48.4	35.6	51.1	63.6	72.5	47.5	68.1	84.8	96.7
0.12	0.50	28.3	40.6	50.5	57.6	42.4	60.9	75.8	86.4	56.6	81.2	101.1	115.2
	0.55	25.7	36.9	45.9	52.4	38.6	55.4	68.9	78.6	51.4	73.8	91.9	104.8
	0.60	23.6	33.8	42.1	48.0	35.4	50.7	63.2	72.0	47.1	67.7	84.2	96.0
	0.65	21.8	31.2	38.9	44.3	32.6	46.8	58.3	66.5	43.5	62.4	77.7	88.6
						Rolling Terrain							
0.08	0.50	38.3	55.0	68.5	78.1	57.5	82.5	102.7	117.1	76.6	110.0	136.9	156.1
	0.55	34.8	50.0	62.2	71.0	52.3	75.0	93.4	106.5	69.7	100.0	124.5	141.9
	0.60	31.9	45.8	57.1	65.1	47.9	68.7	85.6	97.6	63.9	91.7	114.1	130.1
	0.65	29.5	42.3	52.7	60.0	44.2	63.5	79.0	90.1	59.0	84.6	105.3	120.1
0.09	0.50	34.1	48.9	60.9	69.4	51.1	73.3	91.3	104.1	68.1	97.8	121.7	138.8
	0.55	31.0	44.4	55.3	63.1	46.5	66.7	83.0	94.6	61.9	88.9	110.7	126.2
	0.60	28.4	40.7	50.7	57.8	42.6	61.1	76.1	86.7	56.8	81.5	101.4	115.7
	0.65	26.2	37.6	46.8	53.4	39.3	56.4	70.2	80.1	52.4	75.2	93.6	106.8
0.10	0.50	30.7	44.0	54.8	62.5	46.0	66.0	82.2	93.7	61.3	88.0	109.6	124.9
	0.55	27.9	40.0	49.8	56.8	41.8	60.0	74.7	85.2	55.7	80.0	99.6	113.5
	0.60	25.5	36.7	45.6	52.0	38.3	55.0	68.5	78.1	51.1	73.3	91.3	104.1
	0.65	23.6	33.8	42.1	48.0	35.4	50.8	63.2	72.1	47.2	67.7	84.3	96.1
0.11	0.50	27.9	40.0	49.8	56.8	41.8	60.0	74.7	85.2	55.7	80.0	99.6	113.5
	0.55	25.3	36.4	45.3	51.6	38.0	54.5	67.9	77.4	50.7	72.7	90.5	103.2
	0.60	23.2	33.3	41.5	47.3	34.8	50.0	62.2	71.0	46.5	66.7	83.0	94.6
	0.65	21.4	30.8	38.3	43.7	32.2	46.1	57.5	65.5	42.9	61.5	76.6	87.3
0.12	0.50	25.5	36.7	45.6	52.0	38.3	55.0	68.5	78.1	51.1	73.3	91.3	104.1
	0.55	23.2	33.3	41.5	47.3	34.8	50.0	62.2	71.0	46.5	66.7	83.0	94.6
	0.60	21.3	30.6	38.0	43.4	31.9	45.8	57.1	65.1	42.6	61.1	76.1	86.7
	0.65	19.7	28.2	35.1	40.0	29.5	42.3	52.7	60.0	39.3	56.4	70.2	80.1

Note: Key assumptions: 12% trucks, PHF = 0.88, FFS = 60 mi/h.

Appropriate Use of Service Volume Tables

The preceding service volume tables must be used with care. Because the characteristics of any given freeway or multilane highway may or may not be typical, the values should not be used to evaluate a specific freeway or multilane highway segment. The exhibits are intended to allow a general evaluation of many facilities within a given jurisdiction on a first-pass basis to identify segments or facilities that might fail to meet a jurisdiction's operating standards. The segments or facilities so identified should then be evaluated in more detail with this chapter's core methodology in combination with each segment's site-specific characteristics. These service volume tables should not be used to make final decisions on which segments or facilities to improve or on specific designs for such improvements.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for the application of alternative tools to the analysis of basic freeway and multilane highway segments.

Exhibit 12-43 tabulates the HCM limitations for basic freeway and multilane highway segments along with the potential for improved treatment by alternative tools.

Exhibit 12-43
Limitations of HCM Basic Freeway and Multilane Highway Segments Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Special lanes reserved for a single vehicle type, such as truck, and climbing lanes, or specific lane control treatments to restrict lane changing	Modeled explicitly by simulation
Extended bridge and tunnel segments	Can be approximated by using assumptions related to desired speed and number of lanes along each segment
Segments near a toll plaza	Can be approximated by using assumptions related to discharge at toll plaza
Facilities with FFS less than 55 mi/h or more than 75 mi/h for basic freeway segments, or less than 45 mi/h or more than 70 mi/h for multilane highways	Modeled explicitly by simulation
Oversaturated conditions (refer to Chapters 10 and 26 for further discussion)	Modeled explicitly by simulation
Influence of downstream blockages or queuing on a segment	Modeled explicitly by simulation
Posted speed limit and extent of police enforcement	Can be approximated by using assumptions related to desired speed along a segment
Presence of ITS features related to vehicle or driver guidance, and active traffic and demand management strategies, including ramp metering	Several features modeled explicitly by simulation; others may be approximated by using assumptions (for example, by modifying origin–destination demands by time interval)
Evaluation of transition zones where a multilane highway transitions to a two-lane highway or is interrupted by a traffic signal or roundabout intersection	Modeled explicitly by simulation
The negative impacts of poor weather conditions, traffic accidents or incidents, railroad crossings, or construction operations on multilane highways	Limited guidance for modeling adverse conditions on multilane highways in simulation
Differences between types of median barriers and difference between impacts of a median barrier and a TWLTL on multilane highways	Limited guidance available for modeling in simulation
Significant presence of on-street parking, bus stops, and pedestrians on multilane highways	Can be estimated in some simulation tools

As with most other procedural chapters in the HCM, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of downstream conditions on lane utilization and backup beyond the segment boundary is a good example of an analysis that can benefit from the increased insight offered by a microscopic model.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The LOS for basic freeway and multilane highway segments is based on traffic density expressed in passenger cars per mile per lane. The HCM methodology estimates density by dividing the flow rate by the average passenger car speed. Simulation models typically estimate density by dividing the average number of vehicles in the segment by the area of the segment (in lane miles). The result is vehicles per lane mile. This measurement corresponds to density based on space mean speed. The HCM-reported density is also based on space mean speed. Generally, increased speed variability in driver behavior (which simulators usually include) results in lower average space mean speed and higher density. In obtaining density from alternative models, the following are important to take into account:

- The vehicles included in the density estimation (for example, whether only the vehicles that have exited the link are considered);
- The manner in which auxiliary lanes are considered;
- The units used for density, since a simulation package would typically provide density in units of vehicles rather than passenger cars; converting the simulation outputs to passenger cars with the HCM PCE values is typically not appropriate, given that the simulation should already account for the effects of heavy vehicles on a microscopic basis—with heavy vehicles operating at lower speeds and at longer headways—thus making any additional adjustments duplicative;
- The units used in the reporting of density (e.g., whether it is reported per lane mile);
- The homogeneity of the analysis segment, since the HCM does not use the segment length as an input (unless it is a specific upgrade or downgrade segment, where the length is used to estimate the PCE values) and conditions are assumed to be homogeneous for the entire segment; and
- The driver variability assumed in the simulation package, since increased driver variability will generally increase the average density.

The HCM provides capacity estimates in passenger cars per hour per lane as a function of FFS. To compare the HCM's estimates with capacity estimates from a simulation package, the following should be considered:

- The manner in which a simulation package provides the number of vehicles exiting a segment; in some cases it may be necessary to provide virtual detectors at a specific point on the simulated segment so that the maximum throughput can be obtained;
- The units used to specify maximum throughput, since a simulation package would do this in units of vehicles rather than passenger cars; converting vehicles to passenger cars by using the HCM PCE values is typically not appropriate, since differences between automobile and

heavy vehicle performance should already be accounted for microscopically within a simulation; and

- The incorporation of other simulation inputs, such as the "minimum separation of vehicles," that affect the capacity result.

Conceptual Differences Between HCM and Simulation Modeling That Preclude Direct Comparison of Results

The HCM methodology is based on the relationship between speed and flow for various values of FFS. One fundamental potential difference between the HCM and other models is this relationship. For example, the HCM assumes a constant speed for a wide range of flows. However, this is not necessarily the case in simulation packages, some of which assume a continuously decreasing speed with increasing flow. Furthermore, in some simulation packages, that relationship can change when certain parameters (for example, in a car following model) are modified. Therefore, compatibility of performance measures between the HCM and an alternative model for a given set of flows does not necessarily guarantee compatibility for all other sets of flows.

Adjustment of Simulation Parameters to HCM Results

The most important elements to be adjusted when a basic freeway or multilane highway segment is analyzed are the speed–flow relationship, the capacity, or both. The speed–flow relationship should be examined as a function of the given FFS. That FFS should match the field- or HCM-estimated value.

Step-by-Step Recommendations for Applying Alternative Tools

This section provides recommendations specifically for freeway and multilane highway segments (general guidance on selecting and applying simulation packages is provided in Chapter 6, HCM and Alternative Analysis Tools). To apply an alternative tool to the analysis of basic freeway and multilane highway segments, the following steps should be taken:

1. Determine whether the chosen tool can provide density and capacity for a basic freeway or multilane highway segment and the approach used to obtain those values. Once the analyst is satisfied that density and capacity can be obtained and that values compatible with those of the HCM can also be obtained, proceed with the analysis.
2. Determine the FFS of the study site, either from field data or by estimating it according to this chapter's methodology.
3. Enter all available geometric and traffic characteristics into the simulation package and install virtual detectors along the study segment, if necessary, to obtain speeds and flows.
4. By loading the study network over capacity, obtain the maximum throughput and compare it with the HCM estimate. Calibrate the simulation package by modifying parameters related to the minimum time headway so that the capacity obtained by the simulator closely matches the HCM estimate. Estimate the number of runs required for a statistically valid comparison.

5. If the analysis requires evaluating various demand conditions for the segment, plot the simulator's speed–flow curve and compare it with the HCM relationship. Attempt to calibrate the simulation package by modifying parameters related to driver behavior, such as the distribution of driver types. Calibration of the simulation to match the HCM speed–flow relationship may not be possible. In that case, the results should be viewed with caution in terms of their compatibility with the HCM methods.

Sample Calculations Illustrating Alternative Tool Applications

Chapter 26, Freeway and Highway Segments: Supplemental, in Volume 4 of the HCM, provides two supplemental problems that examine situations beyond the scope of this chapter's methodology by using a typical microsimulation-based tool. Both problems analyze a six-lane freeway segment in a growing urban area. The first supplemental problem evaluates the facility when an HOV lane is added, and the second problem analyzes operations with an incident within the segment.

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

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CHAPTER 13
FREEWAY WEAVING SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

Weaving is generally defined as the crossing of two or more traffic streams traveling in the same direction along a significant length of highway without the aid of traffic control devices (except for guide signs). Thus, weaving segments are formed when merge segments are closely followed by diverge segments. "Closely" implies that there is not sufficient distance between the merge and diverge segments for them to operate independently.

Three geometric characteristics affect a weaving segment's operating characteristics: length, width, and configuration. All have an impact on the critical lane-changing activity, which is the unique operating feature of a weaving segment. This chapter provides a methodology for analyzing the operation of weaving segments on the basis of these characteristics as well as a segment's free-flow speed (FFS) and the demand flow rates for each movement within a weaving segment (e.g., ramp to freeway or ramp to ramp).

This chapter describes how the methodology can be applied to planning, operations, and design applications. The methodology can further be used to estimate the effects of weather and incidents on weaving segment computations, and it includes an extension to apply concepts to weaving segments on managed lanes. Example problems are included in Chapter 27, Freeway Weaving: Supplemental.

CHAPTER ORGANIZATION

Chapter 13 presents methodologies for analyzing freeway weaving segment operations in uninterrupted-flow conditions. The chapter presents a methodology for evaluating isolated freeway weaving segments, as well as several extensions to the core method, including analysis of weaving maneuvers on managed lanes.

Section 2 of this chapter presents the following aspects of weaving segments: length and width of a weaving segment, configurations of weaving segments, definitions of key terms used in the methodology, and discussion of special cases.

Section 3 presents the core method for evaluating automobile operations on weaving segments. This method generates the following performance measures:

- Weaving segment capacity;
- Average speed of weaving vehicles, nonweaving vehicles, and all vehicles;
- Average density in the weaving segment; and
- Level of service (LOS) of the weaving segment.

Section 4 extends the core method presented in Section 3 to incorporate considerations for multiple weaving segments, collector–distributor (C-D) roads, and weaving on multilane highways. This section also discusses operational impacts of weaving maneuvers on managed lane facilities.

Section 5 presents guidance on using the results of a freeway weaving segment analysis, including example results from the methods, information on the sensitivity of results to various inputs, and a discussion of service volume tables for weaving segments.

RELATED HCM CONTENT

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

- Chapter 3, Modal Characteristics, discusses general characteristics of the motorized vehicle mode on freeway facilities.
- Chapter 4, Traffic Operations and Capacity Concepts, provides background speed–flow–density concepts of freeway segments that form the basis of weaving concepts presented in this chapter's Section 2.
- Chapter 10, Freeway Facility Core Methodology, provides a method for evaluating weaving segments within an extended freeway facility and their interaction with basic, merge, and diverge segments.
- Chapter 11, Freeway Reliability Analysis, provides a method for evaluating freeway facilities with weaving segments in a reliability context. The chapter also provides default speed and capacity adjustment factors that can be applied in this chapter's methodology.
- Chapter 12, Basic Freeway and Multilane Highway Segments, must be used to evaluate the weaving in segments that exceed the maximum weaving length. For such segments, Chapter 14, Freeway Merge and Diverge Segments, is also used to perform ramp capacity checks.
- Chapter 27, Freeway Weaving: Supplemental, presents example problems and additional methodological details for weaving segments.
- Case Study 4, New York State Route 7, in the *HCM Applications Guide* in Volume 4, demonstrates how this chapter's methods can be applied to the evaluation of an actual freeway facility.
- Section H, Freeway Analyses, in the *Planning and Preliminary Engineering Applications Guide to the HCM,* found in Volume 4, describes how to incorporate this chapter's methods and performance measures into a planning effort.

2. CONCEPTS

OVERVIEW

Exhibit 13-1 illustrates a freeway weaving segment with four principal entry and exit points: A, left entering flow; B, right entering flow; C, left exiting flow; and D, right exiting flow. In many cases, one entry and one exit roadway are ramps, which may be on the right or left side of the freeway mainline. Some weaving segments, however, involve major merge or diverge points at which neither roadway can clearly be labeled a ramp.

On entry and exit roadways, or *legs*, vehicles traveling from Leg A to Leg D must cross the path of vehicles traveling from Leg B to Leg C. Therefore, Flows A–D and B–C are referred to as *weaving movements*. Flows A–C and B–D are not required to cross the path of any other flow and are referred to as *nonweaving movements*.

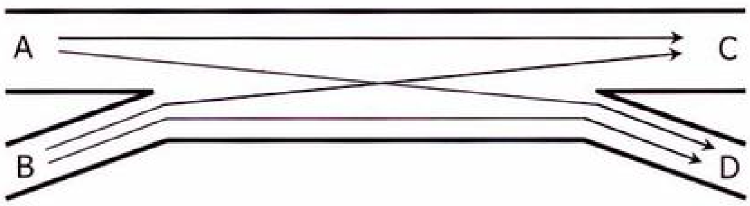


Exhibit 13-1
Formation of a Weaving Segment

Weaving segments require intense lane-changing maneuvers because drivers must access lanes appropriate to their desired exit leg. Therefore, traffic in a weaving segment is subject to lane-changing turbulence in excess of that normally present on basic freeway segments. The added turbulence presents operational problems and design requirements that are addressed by this chapter's methodology.

Traffic in a weaving segment experiences more lane-changing turbulence than is normally present on basic freeway segments.

Three geometric characteristics affect a weaving segment's operating characteristics:

A weaving segment's geometry affects its operating characteristics.

- Length,
- Width, and
- Configuration.

Length is the distance between the merge and diverge that form the weaving segment. *Width* refers to the number of lanes within the weaving segment. *Configuration* is defined by the way entry and exit lanes are aligned. All have an impact on the critical lane-changing activity, which is the unique operating feature of a weaving segment.

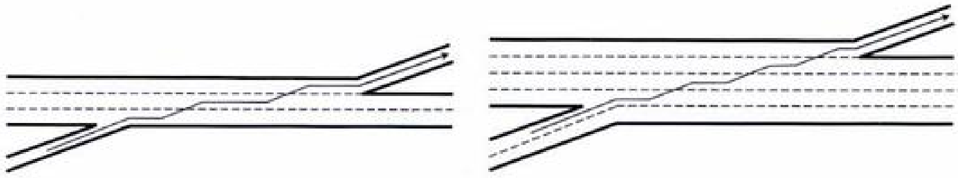
LENGTH OF A WEAVING SEGMENT

The two measures of weaving segment length that are relevant to this chapter's methodology are illustrated in Exhibit 13-2.

ramp) can be made without making a lane change. Again, lane-changing turbulence is focused on the right side of the freeway.

Exhibit 13-4 contains two examples of two-sided weaving segments.

Exhibit 13-4
Two-Sided Weaving Segments Illustrated



(a) Two-Sided Weaving Segment with Single-Lane Ramps

(b) Two-Sided Weaving Segment with Three Lane Changes

Exhibit 13-4(a) is the most common form of a two-sided weave. A one-lane, right-side on-ramp is closely followed by a one-lane, left-side off-ramp (or vice versa). Although the ramp-to-ramp weaving movement requires only two lane changes, this movement is still classified as a two-sided weave because the geometry of the segment features on-ramp and off-ramps on opposite sides of the freeway.

Exhibit 13-4(b) is a less typical case in which one of the ramps has multiple lanes. Because the ramp-to-ramp weaving movement must execute three lane changes, it is also classified as a two-sided weaving segment.

Ramp-Weave and Major Weave Segments

Exhibit 13-3 can also be used to illustrate the difference between a ramp-weaving segment and a major weaving segment. Exhibit 13-3(a) shows a typical ramp-weaving segment, which is defined as follows:

- A *ramp weave* is formed by a one-lane on-ramp closely followed by a one-lane off-ramp, connected by a continuous freeway auxiliary lane.
- The unique feature of the *ramp-weave* configuration is that all weaving drivers must execute a lane change across the lane line separating the freeway auxiliary lane from the right lane of the freeway mainline.

One-sided configurations without a continuous auxiliary lane connecting an on-ramp to a closely following off-ramp are treated as isolated ramp junctions (Chapter 14) and not as weaving segments.

The case of a one-lane on-ramp closely followed by a one-lane off-ramp (on the same side of the freeway), but not connected by a continuous freeway auxiliary lane, is not considered to be a weaving configuration. Such cases are treated as isolated merge and diverge segments and are analyzed with the methodology described in Chapter 14. The distance between the on-ramp and the off-ramp is not a factor in this determination.

Exhibit 13-3(b) shows a typical *major weaving segment,* which is formed when three or more entry or exit legs have multiple lanes. A major weaving segment is distinguished from a major merge or diverge segment in the sense that the latter segments do not feature an auxiliary lane movement between an on-ramp and a downstream off-ramp. A major weave can arise because of a system interchange and connection with another freeway or because of an interchange with an arterial street with multiple lanes on the on-ramp, the off-ramp, or both.

Numerical Measures of Configuration

Three numerical descriptors of a weaving segment characterize its configuration:

LC_{RF} = minimum number of lane changes that a ramp-to-freeway weaving vehicle must make to complete the ramp-to-freeway movement successfully.

LC_{FR} = minimum number of lane changes that a freeway-to-ramp weaving vehicle must make to complete the freeway-to-ramp movement successfully.

N_{WL} = number of lanes from which a weaving maneuver may be completed with one lane change or no lane changes.

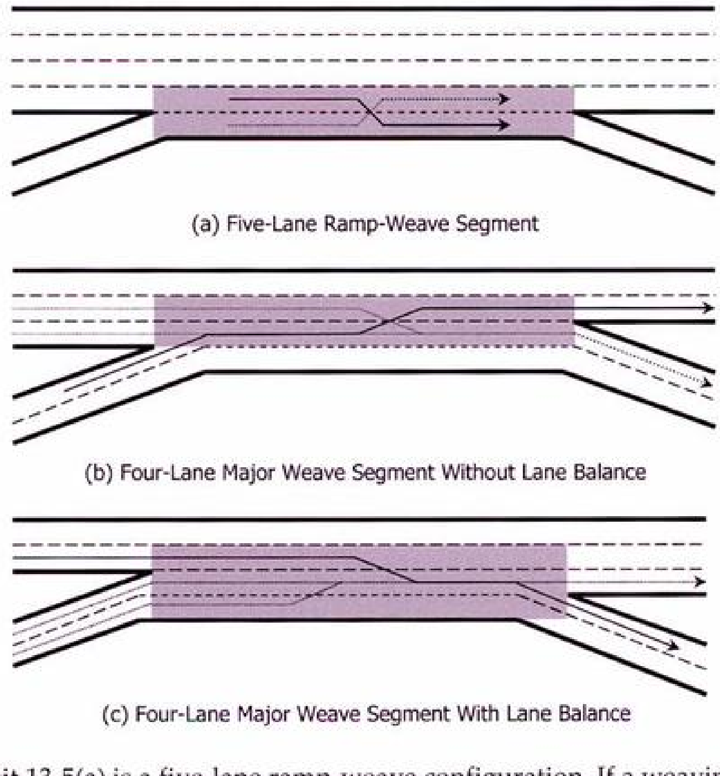
"Minimum number of lane changes" assumes vehicles position themselves when entering and exiting to make the least number of lane changes possible.

These definitions apply directly to one-sided weaving segments in which the ramp-to-freeway and freeway-to-ramp movements are the weaving movements. Different definitions apply to two-sided weaving segments.

Configuration of One-Sided Weaving Segments

Exhibit 13-5 illustrates how these values are determined for one-sided weaving segments. The values of LC_{RF} and LC_{FR} are found by assuming that every weaving vehicle enters the segment in the lane closest to its desired exit leg and leaves the segment in the lane closest to its entry leg.

Exhibit 13-5
Configuration Parameters Illustrated



(a) Five-Lane Ramp-Weave Segment

(b) Four-Lane Major Weave Segment Without Lane Balance

(c) Four-Lane Major Weave Segment With Lane Balance

Exhibit 13-5(a) is a five-lane ramp-weave configuration. If a weaving driver wishes to exit on the off-ramp and enters the segment on the rightmost freeway lane (the lane closest to the off-ramp), the driver must make a single lane change to enter the freeway auxiliary lane and leave via the off-ramp. Thus, for this case,

$LC_{FR} = 1$. A weaving driver entering the freeway via the on-ramp has no choice but to enter on the freeway auxiliary lane. The driver must then make a single lane change from the freeway auxiliary lane to the rightmost lane of the freeway (the lane closest to the entry leg). Thus, $LC_{RF} = 1$ as well.

Lane balance within a weaving segment provides operational flexibility.

Exhibit 13-5(b) and Exhibit 13-5(c) are both major weaving configurations consisting of four lanes. They differ only in the configuration of their entry and exit gore areas. One has lane balance, while the other does not. Lane balance exists when the number of lanes leaving a diverge segment is one more than the number of lanes entering it.

Exhibit 13-5(b) is not typical. It is used here only to demonstrate the concept of lane balance in a major weaving segment. Five lanes approach the entry to the segment and four lanes leave it; four lanes approach the exit from the segment and four lanes leave it. Because of this configuration, vehicles approaching the exit gore must already be in an appropriate lane for their intended exit leg.

In Exhibit 13-5(b), the ramp-to-freeway weaving movement (right to left) requires at least one lane change. A vehicle can enter the segment on the leftmost ramp lane (the lane closest to the desired exit) and make a single lane change to exit on the rightmost lane of the continuing freeway. LC_{RF} for this case is 1. The freeway-to-ramp weaving movement can be made without any lane changes. A vehicle can enter on the rightmost lane of the freeway and leave on the leftmost lane of the ramp without executing a lane change. For this case, $LC_{FR} = 0$.

The exit junction in Exhibit 13-5(c) has lane balance: four lanes approach the exit from the segment and five lanes leave it. This is a desirable feature that provides some operational flexibility. One lane—in this case, the second lane from the right—splits at the exit. A vehicle approaching in this lane can take either exit leg without making a lane change. This is a useful configuration in cases in which the split of exiting traffic varies over a typical day. The capacity provided by the splitting lane can be used as needed by vehicles destined for either exit leg.

In Exhibit 13-5(c), the ramp-to-freeway movement can be made without a lane change, while the freeway-to-ramp movement requires a single lane change. For this case, $LC_{RF} = 0$ and $LC_{FR} = 1$. Ramp-to-freeway vehicles may enter on either of the two lanes of the on-ramp and complete a weaving maneuver with either one or no lane changes. Freeway-to-ramp vehicles may enter on the rightmost freeway lane and also weave with a single lane change. In this case, $N_{WL} = 3$.

In Exhibit 13-5(a), there are only two lanes from which a weaving movement may be made with no more than one lane change. Weaving vehicles may enter the segment in the freeway auxiliary lane (ramp-to-freeway vehicles) and in the rightmost freeway lane (freeway-to-ramp vehicles) and may execute a weaving maneuver with a single lane change. Although freeway-to-ramp vehicles may enter the segment on the outer freeway lanes, they would have to make more than one lane change to access the off-ramp. Thus, for this case, $N_{WL} = 2$.

In Exhibit 13-5(b), weaving vehicles entering the segment in the leftmost lane of the on-ramp or the rightmost lane of the freeway are forced to merge into a single lane. From this lane, the freeway-to-ramp movement can be made with no lane changes, while the ramp-to-freeway movement requires one lane change. Because the movements have merged into a single lane, this counts as one lane from which weaving movements can be made with one or no lane changes. Freeway-to-ramp vehicles, however, may also enter the segment on the center lane of the freeway and make a single lane change (as shown) to execute their desired maneuver. Thus, for this case, N_{WL} is once again 2.

In all one-sided weaving segments, the number of lanes from which weaving maneuvers may be made with one or no lane changes is either two or three. No other values are possible. Segments with $N_{WL} = 3$ generally exist in major weaving segments with lane balance at the exit gore.

Configuration of Two-Sided Weaving Segments

Only the ramp-to-ramp movement is considered to be a weaving flow in a two-sided weaving segment.

The parameters defining the impact of configuration apply only to one-sided weaving segments. In a two-sided weaving segment, neither the ramp-to-freeway nor the freeway-to-ramp movements weave. While the through freeway movement in a two-sided weaving segment might be functionally thought of as weaving, it is the dominant movement in the segment and does not behave as a weaving movement. Thus, in two-sided weaving segments, only the ramp-to-ramp movement is considered to be a weaving flow. This introduces two specific changes to the methodology:

1. Instead of LC_{RF} and LC_{FR} being needed to characterize weaving behavior, a value of LC_{RR} (the minimum number of lane changes that must be made by a ramp-to-ramp vehicle) is needed. In Exhibit 13-4(a), $LC_{RR} = 2$, while in Exhibit 13-4(b), $LC_{RR} = 3$.
2. In all cases of two-sided weaving, the value of N_{WL} is set to 0 by definition.

For cases in which "ramps" cannot be clearly defined, LC_{RR} is the weaving movement requiring three or more lane changes. With these two modifications, the methodology outlined for one-sided weaving segments may be applied to two-sided weaving segments as well.

LOS CRITERIA

The LOS in a weaving segment, as in all freeway analysis, is related to the density in the segment. Exhibit 13-6 provides LOS criteria for weaving segments on freeways, C-D roads, and multilane highways. This methodology was developed for freeway weaving segments, although an isolated C-D roadway was included in its development. The methodology may be applied to weaving segments on uninterrupted segments of multilane surface facilities, although its use in such cases is approximate.

Exhibit 13-6
LOS for Weaving Segments

LOS	Density (pc/mi/ln) Freeway Weaving Segments	Density (pc/mi/ln) Weaving Segments on Multilane Highways or C-D Roads
A	0–10	0–12
B	>10–20	>12–24
C	>20–28	>24–32
D	>28–35	>32–36
E	>35–43	>36–40
F	>43, or demand exceeds capacity	>40, or demand exceeds capacity

The boundary between stable and unstable flow—the boundary between LOS E and F—occurs when the demand flow rate exceeds the capacity of the segment, when density exceeds 43 pc/mi/ln on freeway weaving segments, or when density exceeds 40 pc/mi/ln for weaving segments on multilane highways or C-D roads. The threshold densities for other levels of service were set relative to the criteria for basic freeway segments (or multilane highways). In general, density thresholds in weaving segments are somewhat higher than those for similar basic freeway segments (or multilane highways). Drivers are believed to tolerate higher densities in areas where lane-changing turbulence is expected than on basic segments.

3. CORE METHODOLOGY

The methodology presented in this chapter was developed as part of National Cooperative Highway Research Program (NCHRP) Project 03-75, Analysis of Freeway Weaving Sections (*1*). Elements of this methodology have also been adapted from earlier studies and earlier editions of this manual (*2–9*).

SCOPE OF THE METHODOLOGY

Spatial and Temporal Limits

The methodology of this chapter is based on analysis of the peak 15-min interval within the analysis hour. The analysis hour is most often the peak hour, but the method can be applied to any hour of the day. As in most capacity analysis methodologies, demand flow rates are expressed as hourly equivalent flow rates in vehicles per hour, and not as 15-min volume counts.

The output of the analysis describes operations in all lanes within the defined weaving segment. The influence area of a weaving segment includes the base length of the segment L_B, plus 500 ft upstream and downstream. Research on the operational performance of weaving segments has found that the weave turbulence and associated speed reductions extend beyond the physical (gore-to-gore) boundaries of the weaving segments. This effect is accounted for in the expanded influence area, extending 500 ft on either side of the gore-to-gore distance.

Performance Measures

The procedures described in this chapter result in estimates of the average speed of weaving vehicles S_w, the average speed of nonweaving vehicles S_{nw}, the average speed of all vehicles S, and the average density D within the weaving segment. Average density is used as the service measure for the determination of LOS.

Strengths of the Methodology

The procedures in this chapter were developed from extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent an expert consensus. Most alternative tools will not include the level of detail present in this methodology concerning the weaving configuration and balance of weaving demand flows.

Specific strengths of the HCM procedure include

- Providing capacity estimates for specific weaving configurations as a function of various input parameters, which current alternative tools do not provide directly (and in some cases may require as an input);
- Considering geometric characteristics (such as lane widths) in more detail than most simulation tools;
- Producing a single deterministic estimate of LOS, which is important for some purposes, such as development impact reviews;

- Generating reproducible results with a small commitment of resources (including calibration) from a precisely documented methodology; and
- Evaluating the performance of managed lane (ML) access segments, as well as cross-weaving effects on general purpose lanes due to nearby managed lane access points.

Limitations of the Methodology

The methodology of this chapter does not specifically address the following subjects (without modifications by the analyst):

- Ramp metering on entrance ramps forming part of the weaving segment;
- Segment speed and other performance measure estimation during oversaturated conditions; however, these are addressed in Chapter 10, Freeway Facility Core Methodology;
- Effects of speed limit enforcement practices on weaving segment operations;
- Effects of intelligent transportation system technologies on weaving segment operations;
- Effects of downstream congestion or upstream demand starvation on the analysis segment; however, these are captured by the Chapter 10 methodology;
- Multiple weaving segments, which must be divided into appropriate merge, diverge, and simple weaving segments for analysis; and
- Weaving segments on urban streets and arterials, since urban street weaving is strongly affected by the proximity and timing of signals along the road. At the present time, there are no generally accepted methodologies for analyzing weaving movements on urban streets, including one-way frontage roads.

Multiple weaving segments must be divided into merge, diverge, and simple weaving segments for analysis.

Alternative Tool Consideration

Weaving segments can be analyzed by using a variety of stochastic and deterministic simulation tools that address freeways. These tools can be useful in analyzing the extent of congestion when there are failures within the simulated facility range and when interaction with other freeway segments and other facilities is present.

REQUIRED DATA AND SOURCES

To implement this analysis methodology, demand volumes for each weaving and nonweaving flow must be provided, or hourly flows must be combined with a peak hour factor (PHF), which allows their conversion to flow rates.

A complete geometric description of the weaving segment, including the number and alignment of lanes, lengths, and pavement markings, is also required.

Data can be collected specifically for this purpose. Where detectors exist on entry and exit legs, they may be used to gather volume or flow rate data. Aerial photos can be used to assist in defining the segment geometry.

Exhibit 13-7 lists the information necessary to apply the freeway weaving methodology and suggests potential sources for obtaining these data. It also suggests default values for use when segment-specific information is not available. The user is cautioned that every use of a default value instead of a field-measured, segment-specific value may make the analysis results more approximate and less related to the specific conditions that describe the highway. HCM defaults should only be used when (*a*) field data cannot be collected and (*b*) locally derived defaults do not exist.

Exhibit 13-7
Required Input Data, Potential Data Sources, and Default Values for Freeway Weaving Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Number of lanes	Road inventory, aerial photo	Must be provided
One-sided versus two-sided weave	Road inventory, aerial photo	Must be provided
Short length of weaving segment	Road inventory, aerial photo	Must be provided
Number of lane changes, ramp to freeway	Road inventory, aerial photo	1[a]
Number of lane changes, freeway to ramp	Road inventory, aerial photo	1[a]
Number of lane changes, ramp to ramp	Road inventory, aerial photo	0[a]
Number of weaving lanes	Road inventory, aerial photo	2[a]
Interchange density (interchanges/mi)	Field data, aerial photo	Urban: 0.8/mi Rural: 0.4/mi
Terrain type (level, rolling, specific grade)	Design plans, analyst judgment	Must be provided
Free-flow speed (mi/h)	Direct speed measurements, estimate from design speed or speed limit	Speed limit + 5 mi/h
Equivalent capacity of basic freeway segment	Estimated from free-flow speed and Chapter 12	Must be provided
	Demand Data	
Hourly demand volume, freeway to freeway (veh/h)	Field data, modeling	Must be provided[b]
Hourly demand volume, freeway to ramp (veh/h)	Field data, modeling	Must be provided[b]
Hourly demand volume, ramp to freeway (veh/h)	Field data, modeling	Must be provided[b]
Hourly demand volume, ramp to ramp (veh/h)	Field data, modeling	Must be provided[b]
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor[c] (decimal)	Field data	0.94 urban and rural
Speed and capacity adjustment factors for driver population[d]	Field data	1.0
Speed and capacity adjustment factors for weather and incidents[e]	Field data	1.0
Heavy vehicle percentage (%)	Field data	5% urban, 12% rural[f]

Notes: ***Bold italic*** indicates high sensitivity (>20% change) of service measure to the choice of default value.
Bold indicates moderate sensitivity (10%–20% change) of service measure to the choice of default value.
[a] Applicable for weaves with a single-lane on-ramp and single-lane off-ramp, both on right side of road.
[b] A proportional distribution can be assumed from segment entering and exiting volumes.
[c] Moderate to high sensitivity of service measures for very low PHF values. See the discussion in the text. PHF is not required when peak 15-min demand volumes are provided.
[d] See Chapter 26 in Volume 4 for default adjustment factors for driver population.
[e] See Chapter 11 for default capacity and speed adjustment factors for weather and incidents.
[f] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.

The exhibit distinguishes between urban and rural conditions for certain defaults. The classification of a facility into urban and rural is made on the basis of the Federal Highway Administration smoothed or adjusted urbanized boundary definition (*10*), which in turn is derived from Census data.

Care should be taken in using default values. The service measure results are sensitive to some of the input data listed in Exhibit 13-7. For example, the numbers of lane changes from freeway to ramp, ramp to freeway, and ramp to ramp, as well as the number of weaving lanes, all change the service measure result by more than 20% when these inputs are varied over their normal range. In addition, the free-flow speed results in a 10%–20% change in service measure when it is varied over its normal range. A very low PHF value (0.60) results in a greater than 20% change, compared with the results obtained for the default value for PHF; more typical PHFs vary the service measure results by less than 10%. Other inputs change the service measure result by less than 10% when they are varied over their normal range.

OVERVIEW OF THE METHODOLOGY

Models Used by the Methodology

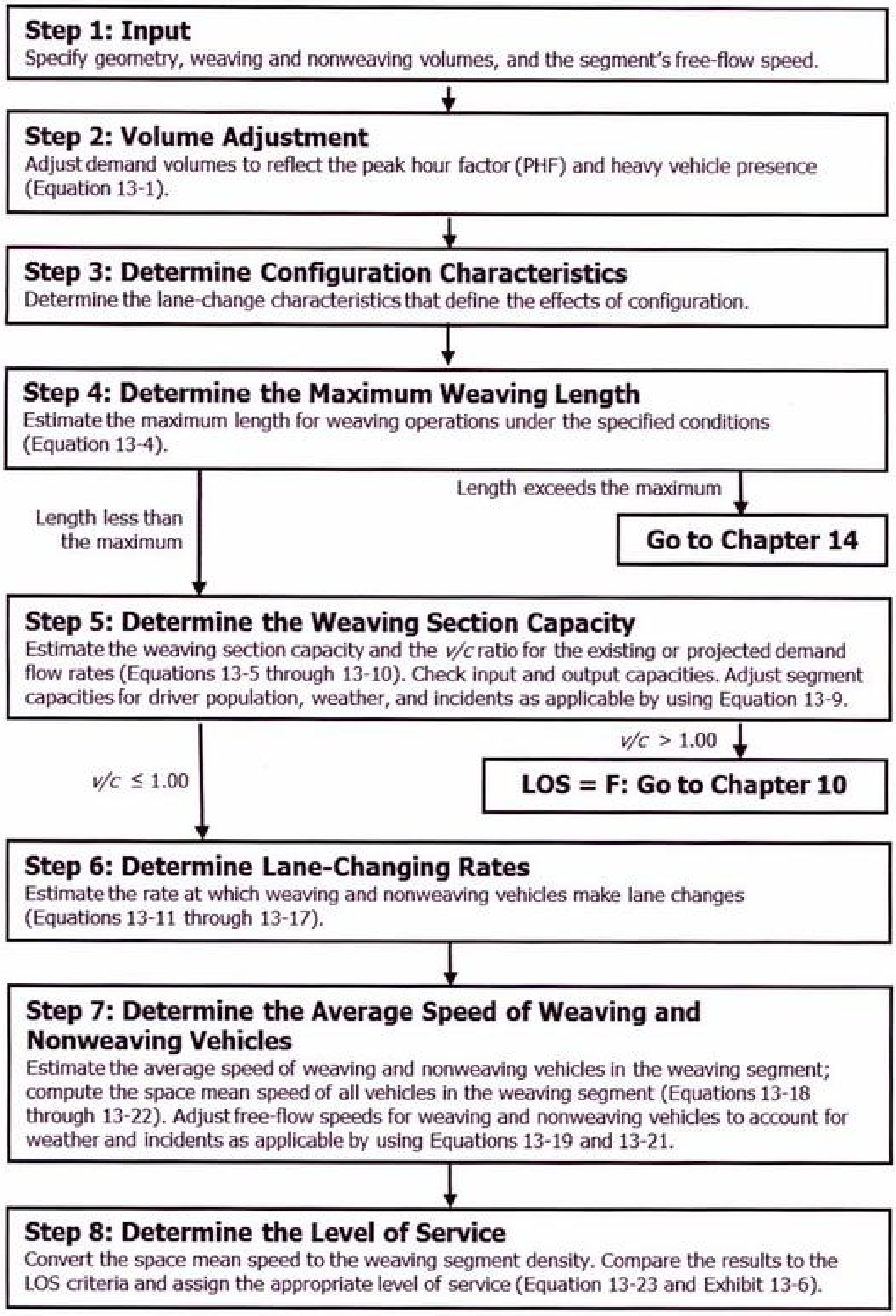
Exhibit 13-8 is a flowchart illustrating the basic steps that define the methodology for analyzing freeway weaving segments. The methodology uses several types of predictive algorithms, all of which are based on a mix of theoretical and regression models. These models include the following:

- Models that predict the total rate of lane changing taking place in the weaving segment. This is a direct measure of turbulence in the traffic stream caused by the presence of weaving movements.
- Models to predict the average speed of weaving and nonweaving vehicles in a weaving segment under stable operating conditions, that is, not operating at LOS F, including adjustments to account for the impacts of weather and incidents.
- Models to predict the capacity of a weaving segment under both ideal and prevailing conditions, including adjustments to account for the impacts of weather and incidents.
- A model to estimate the maximum length over which weaving operations can be said to exist.

Parameters Describing a Weaving Segment

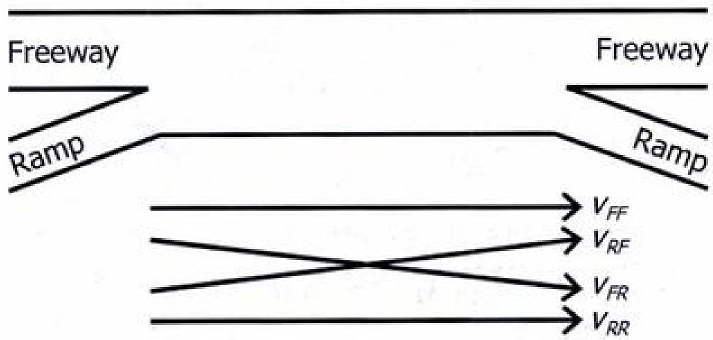
Several parameters describing weaving segments have already been introduced and defined. Exhibit 13-9 illustrates all variables that must be specified as input variables and defines those that will be used within or as outputs of the methodology. Some of them apply only to one-sided weaving segments. Exhibit 13-10 lists the variables that are different in applications to two-sided weaving segments.

Exhibit 13-8
Weaving Methodology Flowchart



LOS F exists in a weaving segment when demand exceeds capacity.

Exhibit 13-9
Weaving Variables for One-Sided Weaving Segments



v_{FF} = freeway-to-freeway demand flow rate in the weaving segment in passenger cars per hour (pc/h);

v_{RF} = ramp-to-freeway demand flow rate in the weaving segment (pc/h);

v_{FR} = freeway-to-ramp demand flow rate in the weaving segment (pc/h);

v_{RR} = ramp-to-ramp demand flow rate in the weaving segment (pc/h);

v_W = weaving demand flow rate in the weaving segment (pc/h), $v_W = v_{RF} + v_{FR}$;

v_{NW} = nonweaving demand flow rate in the weaving segment (pc/h), $v_{NW} = v_{FF} + v_{RR}$;

v = total demand flow rate in the weaving segment (pc/h), $v = v_W + v_{NW}$;

VR = volume ratio (decimal), v_W/v;

N = number of lanes within the weaving segment (ln);

N_{WL} = number of lanes from which a weaving maneuver may be made with one or no lane changes (see Exhibit 13-5) (ln);

S_W = average speed of weaving vehicles within the weaving segment (mi/h);

S_{NW} = average speed of nonweaving vehicles within the weaving segment (mi/h);

S = average speed of all vehicles within the weaving segment (mi/h);

FFS = free-flow speed of the weaving segment (mi/h);

D = average density of all vehicles within the weaving segment in passenger cars per mile per lane (pc/mi/ln);

W = weaving intensity factor;

L_S = length of the weaving segment (ft), based on the short length definition of Exhibit 13-2;

LC_{RF} = minimum number of lane changes (lc) that must be made by a single weaving vehicle moving from the on-ramp to the freeway (see Exhibit 13-5);

LC_{FR} = minimum number of lane changes that must be made by a single weaving vehicle moving from the freeway to the off-ramp (lc);

LC_{MIN} = minimum rate of lane changing that must exist for all weaving vehicles to complete their weaving maneuvers successfully, in lane changes per hour (lc/h), $LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$;

LC_W = total rate of lane changing by weaving vehicles within the weaving segment (lc/h);

LC_{NW} = total rate of lane changing by nonweaving vehicles within the weaving segment (lc/h);

LC_{ALL} = total rate of lane changing of all vehicles within the weaving segment (lc/h), $LC_{ALL} = LC_W + LC_{NW}$;

ID = interchange density, the number of interchanges within 3 mi upstream and downstream of the center of the subject weaving segment divided by 6, in interchanges per mile (int/mi); and

I_{LC} = lane-changing intensity, LC_{ALL}/L_S, in lane changes per foot (lc/ft).

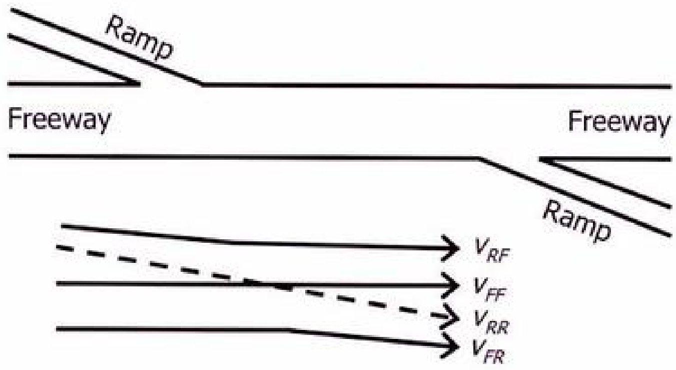


Exhibit 13-10
Weaving Variables for a Two-Sided Weaving Segment

The through freeway movement is not considered to be weaving in a two-sided weaving segment.

All variables are defined as in Exhibit 13-9, except for the following variables relating to flow designations and lane-changing variables:

v_W = total weaving demand flow rate within the weaving segment (pc/h), $v_W = v_{RR}$;

v_{NW} = total nonweaving demand flow rate within the weaving segment (pc/h), $v_{NW} = v_{FR} + v_{RF} + v_{FF}$;

LC_{RR} = minimum number of lane changes that must be made by one ramp-to-ramp vehicle to complete a weaving maneuver; and

LC_{MIN} = minimum rate of lane changing that must exist for all weaving vehicles to complete their weaving maneuvers successfully (lc/h), $LC_{MIN} = LC_{RR} \times v_{RR}$.

The principal difference between one-sided and two-sided weaving segments is the relative positioning of the movements within the segment. In a two-sided weaving segment, the ramp-to-freeway and freeway-to-ramp vehicles do not weave. In a one-sided segment, they execute the weaving movements. In a two-sided weaving segment, the ramp-to-ramp vehicles must cross the path of freeway-to-freeway vehicles. Both could be taken to be weaving movements. In reality, the through freeway movement is not weaving in that vehicles do not need to change lanes and generally do not shift lane position in response to a desired exit leg.

Thus, in two-sided weaving segments, only the ramp-to-ramp flow is considered to be weaving. The lane-changing parameters reflect this change in the way weaving flows are viewed. Thus, the minimum rate of lane changing that weaving vehicles must maintain to complete all desired weaving maneuvers successfully is also related only to the ramp-to-ramp movement.

The methodology uses demand flow rates for the peak 15 min in passenger cars per hour.

The definitions for flow all refer to *demand flow rate*. This means that for existing cases, the demand should be based on *arrival flows*. For future cases, forecasting techniques will generally produce a *demand volume* or *demand flow rate*. All of the methodology's algorithms use demand expressed as flow rates in the peak 15 min of the design (or analysis) hour, in equivalent passenger car units.

COMPUTATIONAL STEPS

Each of the major procedural steps noted in Exhibit 13-8 is discussed in detail in the sections that follow.

Step 1: Input Data

The methodology for weaving segments is structured for operational analysis usage, that is, given a known or specified geometric design and traffic demand characteristics, the methodology is used to estimate the expected LOS.

Design and preliminary engineering are generally conducted in terms of comparative analyses of various design proposals. This is a good approach, given that the range of widths, lengths, and configurations in any given case is constrained by a number of factors. Length is constrained by the location of the crossing arteries that determine the location of interchanges and ramps. Width is constrained by the number of lanes on entry and exit legs and usually involves no more than two choices. Configuration is also the result of the number of lanes on entry and exit legs as well as the number of lanes within the segment. Changing the configuration usually involves adding a lane to one of the entry or exit legs, or both, to create different linkages.

For analysis, the geometry of the weaving segment must be fully defined. This includes the number of lanes, lane widths, shoulder clearances, the details of entry and exit gore area designs (including markings), the existence and extent of barrier lines, and the length of the segment. A sketch of the weaving segment should be drawn with all appropriate dimensions shown.

Step 2: Adjust Volume

All equations in this chapter use flow rates under equivalent ideal conditions as input variables. Thus, demand volumes and flow rates under prevailing conditions must be converted to their ideal equivalents by using Equation 13-1:

Equation 13-1

$$v_i = \frac{V_i}{PHF \times f_{HV}}$$

where

v_i = flow rate i under ideal conditions (pc/h),

V_i = hourly volume for flow i under prevailing conditions in vehicles per hour (veh/h),

PHF = peak hour factor (decimal), and

f_{HV} = adjustment factor for heavy vehicle presence (decimal).

The subscript for the type of flow i can take on the following values:

$_{FF}$ = freeway to freeway, $_{FR}$ = freeway to ramp,

$_{RF}$ = ramp to freeway, $_{RR}$ = ramp to ramp,

$_{W}$ = weaving, and $_{NW}$ = nonweaving.

The heavy vehicle adjustment factor f_{HV} is taken from Chapter 12, Basic Freeway and Multilane Highway Segments.

If flow rates for a 15-min period have been provided as inputs, the PHF is taken to be 1.00 in this computation, and the 15-min count is used directly after conversion to an hourly flow rate.

Once demand flow rates have been established, it may be convenient to construct a weaving diagram similar to those illustrated in Exhibit 13-9 (for one-sided weaving segments) and Exhibit 13-10 (for two-sided weaving segments).

Step 3: Determine Configuration Characteristics

Several key parameters characterize the configuration of a weaving segment. They are descriptive of the segment and will be used as variables in subsequent steps of the methodology:

LC_{MIN} = minimum rate at which weaving vehicles must change lanes to complete all weaving maneuvers successfully (lc/h), and

N_{WL} = number of lanes from which weaving maneuvers may be made with either one or no lane changes (ln).

How these values are determined depends on whether the segment under study is a one-sided or a two-sided weaving segment.

One-Sided Weaving Segments

The determination of key variables in one-sided weaving segments is illustrated in Exhibit 13-9. In one-sided segments, the two weaving movements are the ramp-to-freeway and freeway-to-ramp flows. As shown in Exhibit 13-9, the following values are established:

LC_{RF} = minimum number of lane changes that must be made by one ramp-to-freeway vehicle to execute the desired maneuver successfully (lc), and

LC_{FR} = minimum number of lane changes that must be made by one freeway-to-ramp vehicle to execute the desired maneuver successfully (lc).

LC_{MIN} for one-sided weaving segments is given by Equation 13-2:

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

Equation 13-2

For one-sided weaving segments, the value of N_{WL} is either 2 or 3. The determination is made by a review of the geometric design and the configuration of the segment, as illustrated in Exhibit 13-5.

Two-Sided Weaving Segments

The determination of key variables in two-sided weaving segments is illustrated in Exhibit 13-10. The unique feature of two-sided weaving segments is that only the ramp-to-ramp flow is functionally weaving. From Exhibit 13-10, the following value is established:

LC_{RR} = minimum number of lane changes that must be made by one ramp-to-ramp vehicle to execute the desired maneuver successfully (lc).

LC_{MIN} for two-sided weaving segments is given by Equation 13-3:

Equation 13-3

$$LC_{MIN} = LC_{RR} \times v_{RR}$$

For two-sided weaving segments, the value of N_{WL} is always 0 by definition.

Step 4: Determine Maximum Weaving Length

The maximum length of a weaving segment, L_{MAX}, is based on the distance beyond which additional length does not add to capacity.

The concept of maximum length of a weaving segment is critical to the methodology. Strictly defined, *maximum length* is the length at which weaving turbulence no longer has an impact on operations within the segment, or alternatively, on the capacity of the weaving segment.

Unfortunately, depending on the selected definition, these measures can differ significantly. Weaving turbulence will affect operations (i.e., weaving and nonweaving vehicle speeds) far beyond the point where the segment's capacity is no longer affected by weaving.

This methodology uses the second definition (based on the equivalence of capacity). If the operational definition were used, the methodology would produce capacity estimates in excess of those for a similar basic freeway segment, which is illogical. The maximum length of a weaving segment (in feet) is computed from Equation 13-4:

Equation 13-4

$$L_{MAX} = [5{,}728(1 + VR)^{1.6}] - (1{,}566N_{WL})$$

where L_{MAX} is the maximum weaving segment length in feet (using the short length definition) and other variables are as previously defined.

As VR increases, the influence of weaving turbulence is expected to extend for longer distances. All values of N_{WL} are either 0 (two-sided weaving segments) or 2 or 3 (one-sided weaving segments). Having more lanes from which easy weaving lane changes can be made reduces turbulence, which in turn reduces the distance over which such turbulence affects segment capacity.

Exhibit 13-11 illustrates the sensitivity of maximum length to both VR and N_{WL}. As expected, VR has a significant impact on maximum length, as does the configuration, as indicated by N_{WL}. While the maximum lengths shown can compute to very high numbers, the highest results are well outside the calibration range of the equation (limited to about 2,800 ft), and many of the situations are improbable. Values of VR on segments with N_{WL} = 2.0 lanes rarely rise above the range of 0.40 to 0.50. Values of VR above 0.70 are technically feasible on segments with N_{WL} = 3.0 lanes, but they are rare.

While the extreme values in Exhibit 13-11 are not practical, the maximum length of weaving segments can clearly rise to 6,000 ft or more. Furthermore, the maximum length can vary over time, since VR is not a constant throughout every demand period of the day.

Exhibit 13-11
Variation of Weaving Length Versus Volume Ratio and Number of Weaving Lanes (ft)

VR	Maximum Weaving Length (ft) $N_{WL} = 2$	$N_{WL} = 3$
0.1	3,540	1,974
0.2	4,536	2,970
0.3	5,584	4,018
0.4	6,681	5,115
0.5	7,826	6,260
0.6	9,019	7,453
0.7	10,256	8,690
0.8	11,538	9,972

The value of L_{MAX} is used to determine whether continued analysis of the configuration as a weaving segment is justified:

- If $L_S < L_{MAX}$, continue to Step 5; or
- If $L_S \geq L_{MAX}$, analyze the merge and diverge junctions as separate segments by using the methodology in Chapter 14.

If the length of the segment is greater than L_{MAX}, it should be analyzed as separate merge and diverge ramp junctions by using the methodology in Chapter 14. Any portion falling outside the influence of the merge and diverge segments is treated as a basic freeway segment. In these cases, Chapter 14 performs ramp capacity checks for those segments; however, merge and diverge segments with a continuous lane add or lane drop are eventually analyzed operationally as a basic freeway segment with the procedures in Chapter 12.

If the segment is too long to be considered a weaving segment, the merge and diverge areas are treated separately. In these cases, Chapter 14 performs ramp capacity checks for those segments; however, merge and diverge segments with a continuous lane add or lane drop are eventually analyzed operationally as a basic freeway segment with the procedures in Chapter 12. Any distance falling outside the influence areas of the merge and diverge segments would be considered to be a basic freeway segment and analyzed accordingly.

Step 5: Determine Weaving Segment Capacity

A weaving segment's capacity is controlled by either (a) the average vehicle density reaching 43 pc/mi/ln or (b) the weaving demand flow rate exceeding a value that depends on the number of weaving lanes.

The capacity of a weaving segment is controlled by one of two conditions:

1. Breakdown of a weaving segment is expected to occur when the average density of all vehicles in the segment reaches 43 pc/mi/ln; or
2. Breakdown of a weaving segment is expected to occur when the total weaving demand flow rate exceeds
    - 2,400 pc/h for cases in which $N_{WL} = 2$ lanes, or
    - 3,500 pc/h for cases in which $N_{WL} = 3$ lanes.

The first condition is based on the criteria listed in Chapter 12, Basic Freeway and Multilane Highway Segments, which state that breakdowns occur at a density of 45 pc/mi/ln. Given the additional turbulence in a weaving segment, breakdown is expected to occur at slightly lower densities.

The second condition recognizes that there is a practical limit to how many vehicles can cross each other's path without causing serious operational failures. The existence of a third lane from which weaving maneuvers can be made with two or fewer lane changes in effect spreads the impacts of turbulence across segment lanes and allows for higher weaving flows.

The first criterion is partially a function of the segment length, with longer weaving segments resulting in an increase in segment capacity. However, if capacity is controlled by the weave configuration (i.e., the second criterion applies), then capacity is not dependent on length, since the flow is limited by the configuration of weaving lanes. In this case, lengthening the weaving segment will have no effect on its capacity and the weaving configuration will need to be changed instead.

For two-sided weaving segments (N_{WL} = 0 lanes), no limiting value on weaving flow rate is given. The analysis of two-sided weaving segments is approximated by this methodology, and a density sufficient to cause a breakdown is typically reached at relatively low weaving flow rates. An increase in the length of a two-sided weaving segment generally increases its capacity, since weaving maneuvers are spread over a longer distance.

Weaving Segment Capacity Determined by Density

The capacity of a weaving segment, based on reaching a density of 43 pc/mi/ln, is estimated by using Equation 13-5:

Equation 13-5

$$c_{IWL} = c_{IFL} - [438.2(1 + VR)^{1.6}] + (0.0765L_S) + (119.8N_{WL})$$

where

c_{IWL} = capacity (per lane) of the weaving segment under equivalent ideal conditions (pc/h/ln), and

c_{IFL} = capacity (per lane) of a basic freeway segment with the same FFS as the weaving segment under equivalent ideal conditions (pc/h/ln).

All other variables are as previously defined.

The model describes the capacity of a weaving segment in terms of the difference between the capacity of a basic freeway segment and the capacity of a weaving segment with the same FFS. Capacity decreases with *VR*, which is logical. It increases as length and number of weaving lanes N_{WL} increase. These are also logical trends, since both increasing length and a larger number of weaving lanes reduce the intensity of turbulence.

Arithmetically, a result in which c_{IWL} is greater than c_{IFL} is possible. In practice, this will never occur. The maximum length algorithm of Step 4 was found by setting the two values equal. Thus, weaving analyses would only be undertaken in cases in which c_{IWL} is less than c_{IFL}.

The value of c_{IWL} must now be converted to a total capacity under prevailing conditions by using Equation 13-6:

Equation 13-6

$$c_W = c_{IWL} \times N \times f_{HV}$$

where c_W is the capacity of the weaving segment under prevailing conditions in vehicles per hour. It is stated as a flow rate for a 15-min analysis period, as are all capacities.

Weaving Segment Capacity Determined by Weaving Demand Flows

The capacity of a weaving segment, as controlled by the maximum weaving flow rates noted previously, is found from Equation 13-7:

$$c_{IW} = \begin{cases} \dfrac{2{,}400}{VR} & \text{for } N_{WL} = 2 \text{ lanes} \\ \dfrac{3{,}500}{VR} & \text{for } N_{WL} = 3 \text{ lanes} \end{cases}$$

Equation 13-7

where c_{IW} is the capacity of all lanes in the weaving segment under ideal conditions in passenger cars per hour and all other variables are as previously defined. This value is converted to prevailing conditions with Equation 13-8:

$$c_W = c_{IW} \times f_{HV}$$

Equation 13-8

Determination of Capacity

The final capacity is the smaller of the two estimates of Equation 13-6 and Equation 13-8. Note that this is the expected capacity, in vehicles per hour, for the existing conditions assuming that there are no adverse weather conditions or incidents.

Adjustment to Capacity for Adverse Weather, Incidents, or Driver Population

The capacity of the weaving segment may be further adjusted to account for the impacts of adverse weather, driver population, occurrence of traffic incidents, or a combination of these factors. The methodology for making such adjustments is the same as that for other types of freeway segments. Default adjustment factors are found in Section 5 of Chapter 11, Freeway Reliability Analysis. The adjustments for weather and incidents are most commonly applied in the context of a reliability analysis. For convenience, a brief summary is provided here.

The capacity of a weaving segment is adjusted as shown in Equation 13-9:

$$c_{wa} = c_w \times CAF$$

Equation 13-9

where

c_{wa} = adjusted capacity of weaving area (veh/h),

c_w = unadjusted capacity of weaving area (veh/h), and

CAF = capacity adjustment factor from Chapter 11 (unitless).

The CAF can have several components, including weather, incident, work zone, driver population, and calibration adjustments. CAF defaults for weather and incident effects are found in Chapter 11, along with additional discussion on how to apply them. If desired, capacity can be further adjusted to account for unfamiliar drivers in the traffic stream. While the default CAF for this effect is set to 1.0, Chapter 26 provides guidance for estimating the CAF on the basis of the composition of the driver population.

Chapter 12 provides additional guidance on capacity definitions, and Chapter 26 provides guidance on estimating freeway segment capacity, including weaving segment capacity, from field data.

Volume-to-Capacity Ratio

With the final capacity determined, the volume-to-capacity ratio (v/c ratio) for the weaving segment may be computed from Equation 13-10. The total volume v in this case represents the sum of weaving and nonweaving flows.

Equation 13-10

$$v/c = \frac{v \times f_{HV}}{c_{wa}}$$

The heavy vehicle adjustment factors are used because the total demand flow rate v is stated for equivalent ideal conditions, while c_{wa} is stated for prevailing conditions.

Level of Service F

LOS F occurs when demand exceeds capacity. The methodologies in Chapter 10 can be used to evaluate oversaturated weaving segments.

If v/c is greater than 1.00, demand exceeds capacity, and the segment is expected to fail, that is, have a LOS of F. If this occurs, the analysis is terminated, and LOS F is assigned. At LOS F, queues are expected to form within the segment, possibly extending upstream beyond the weaving segment itself. Queuing on the on-ramps that are part of the weaving segment would also be expected. The analyst is urged to use the methodology of Chapters 10 and 11, on freeway facilities, to analyze the impacts of the existence of LOS F on upstream and downstream segments during the analysis period and over time.

Checking Input and Output Capacities

In most cases, the controlling capacity factor in a weaving segment is the weaving activity itself. The computational procedure for capacity of the weaving segment *guarantees* that the result will be less than the capacity of a basic freeway segment with the same number of lanes. Thus, the conduct of a basic freeway capacity check on the weaving segment itself is not necessary.

In rare cases, there may be insufficient capacity to accommodate the demand flows on one or more of the entry and exit roadways. Input and output roadways must be classified as either basic freeway lanes or ramps. The capacity of basic freeway lanes is checked by using the procedures of Chapter 12, Basic Freeway and Multilane Highway Segments. Ramp capacities should be checked by using the methodology of Chapter 14, Freeway Merge and Diverge Segments.

If either an entry roadway or an exit roadway has insufficient capacity, the weaving segment will not function properly, and queuing resulting from the capacity deficiency will result. LOS F is assigned, and further analysis must use the methodology of Chapters 10 and 11 for freeway facilities.

Step 6: Determine Lane-Changing Rates

The equivalent hourly rate at which weaving and nonweaving vehicles make lane changes within the weaving segment is a direct measure of turbulence. It is also a key determinant of speeds and densities within the segment, which ultimately govern the existing or anticipated LOS.

The lane-changing rates estimated are in terms of equivalent *passenger car lane changes*. Heavy vehicle lane changes are assumed to create more turbulence than passenger car lane changes.

Three types of lane changes can be made within a weaving segment:

- *Required lane changes made by weaving vehicles*: These lane changes must be made to complete a weaving maneuver and are restricted to the physical area of the weaving segment. In Step 3, the rate at which such lane changes are made by weaving vehicles, LC_{MIN}, was determined.

- *Optional lane changes made by weaving vehicles*: These lane changes are not necessary to weave successfully. They involve weaving drivers who choose to enter the weaving segment in the outer lanes of either the freeway or the ramp (assuming it has more than one lane), leave the weaving segment in an outer lane, or both. Such drivers make additional lane changes beyond those absolutely required by their weaving maneuver.
- *Optional lane changes made by nonweaving vehicles*: Nonweaving vehicles may also make lane changes within the weaving segment, but neither the configuration nor their desired origin and destination would require such lane changes. Lane changes by nonweaving vehicles are always made because the driver chooses that option.

While LC_{MIN} can be computed from the weaving configuration and the demand flow rates, additional optional lane changes made by both weaving and nonweaving vehicles add to turbulence and must be estimated by using regression-based models.

Estimating the Total Lane-Changing Rate for Weaving Vehicles

The model for predicting the total lane-changing rate for weaving vehicles is of the form LC_{MIN} plus an algorithm that predicts the additional optional lane-changing rate. These are combined so that the total lane-changing rate for weaving vehicles, including both required and optional lane changes, is as shown in Equation 13-11:

$$LC_W = LC_{MIN} + 0.39[(L_S - 300)^{0.5} N^2 (1 + ID)^{0.8}]$$

Equation 13-11

where

LC_W = equivalent hourly rate at which weaving vehicles make lane changes within the weaving segment (lc/h);

LC_{MIN} = minimum equivalent hourly rate at which weaving vehicles must make lane changes within the weaving segment to complete all weaving maneuvers successfully (lc/h);

L_S = length of the weaving segment, using the short length definition (ft) (300 ft is the minimum value);

N = number of lanes within the weaving segment (ln); and

ID = interchange density, the number of interchanges within 3 mi upstream and downstream of the center of the subject weaving segment divided by 6, in interchanges per mile (int/mi).

Equation 13-11 has several interesting characteristics. The term $L_S - 300$ implies that for weaving segments of 300 ft (or shorter), weaving vehicles only make necessary lane changes, that is, $LC_W = LC_{MIN}$. While shorter weaving segments would be an aberration, they do occasionally occur. However, in applying the equation to short weaving segments, a length of 300 ft is used for all lengths less than or equal to 300 ft.

This model is also unique in that it uses the concept of interchange density, as opposed to total ramp density. The FFS for freeway segments is partially based on total ramp density rather than interchange density. The two measures are, of course, related to the type of interchange involved. A full cloverleaf interchange has four ramps, while a diamond interchange has two ramps. Care must be taken in determining the value of *total ramp density* and *interchange density*, since they are distinct.

The algorithm uses the term $1 + ID$ because the value of ID may be more than or less than 1.00, and the power term would not act consistently on the result. In determining interchange density for a weaving segment, a distance of 3 mi upstream and 3 mi downstream of the midpoint of the weaving segment is used. The number of interchanges within the 6-mi range defined above is counted and divided by 6 to determine the interchange density. If two closely spaced ramps from different cross-streets effectively function as one, they can be counted as a single interchange in the determination of interchange density on the basis of analyst judgment. For additional discussion of total ramp density, consult Chapter 12.

The basic sensitivities of this model are reasonable. Weaving-vehicle lane changing increases as the length and width of the weaving segment increase. A longer, wider weaving segment simply provides more opportunities for weaving vehicles to execute lane changes. Lane changing also increases as interchange density increases. Higher interchange densities mean that there are more reasons for drivers to make optional lane changes based on their entry or exit at a nearby interchange.

Estimating the Lane-Changing Rate for Nonweaving Vehicles

No nonweaving driver *must* make a lane change within the confines of a weaving segment. All nonweaving vehicle lane changes are, therefore, optional. They are more difficult to predict than weaving lane changes, since the motivation for nonweaving lane changes varies widely and may not always be obvious. Such lane changes may be made to avoid turbulence, to be better positioned for a subsequent maneuver, or simply to achieve a higher average speed.

The research leading to this methodology (*1*) revealed several discontinuities in the lane-changing behavior of nonweaving vehicles within weaving segments. To identify the areas of discontinuity and to develop an estimation model for these areas, it was necessary to define a "nonweaving vehicle index," I_{NW}, as given in Equation 13-12:

Equation 13-12

$$I_{NW} = \frac{L_S \times ID \times v_{NW}}{10{,}000}$$

This index is a measure of the tendency of conditions to induce unusually large nonweaving vehicle lane-changing rates. Large nonweaving flow rates, high interchange densities, and long weaving lengths appear to produce situations in which nonweaving lane-changing rates are unusually elevated.

Two models are used to predict the rate at which nonweaving vehicles change lanes in weaving segments. The first, Equation 13-13, covers the majority of cases, that is, cases for which normal lane-changing characteristics are expected. This is the case when I_{NW} is less than or equal to 1,300:

$$LC_{NW1} = (0.206v_{NW}) + (0.542L_S) - (192.6N)$$

Equation 13-13

where LC_{NW1} is the rate of lane changing per hour. The equation shows logical trends in that nonweaving lane changes increase with both nonweaving flow rate and segment length. Less expected is that nonweaving lane changing *decreases* with increasing number of lanes. This trend is statistically very strong and likely indicates more presegregation of flows in wider weaving segments. Arithmetically, Equation 13-13 can produce a negative result. Thus, the minimum value must be externally set at 0.

The second model applies to a small number of cases in which the combination of high nonweaving demand flow, high interchange density, and long segment length produces extraordinarily high nonweaving lane-changing rates. Equation 13-14 is used in cases for which I_{NW} is greater than or equal to 1,950:

$$LC_{NW2} = 2{,}135 + 0.223(v_{NW} - 2{,}000)$$

Equation 13-14

where LC_{NW2} is the lane-changing rate per hour and all other variables are as previously defined.

Unfortunately, Equation 13-13 and Equation 13-14 are discontinuous and cover discontinuous ranges of I_{NW}. If the nonweaving index is between 1,300 and 1,950, a straight interpolation between the values of LC_{NW1} and LC_{NW2} is used as shown in Equation 13-15:

$$LC_{NW3} = LC_{NW1} + (LC_{NW2} - LC_{NW1})\left(\frac{I_{NW} - 1{,}300}{650}\right)$$

Equation 13-15

where LC_{NW3} is the lane-changing rate per hour and all other variables are as previously defined. Equation 13-15 only works for cases in which LC_{NW1} is less than LC_{NW2}. In the vast majority of cases, this will be true (unless the weaving length is longer than the maximum length estimated in Step 4). In the rare case when it is not true, LC_{NW2} is used.

Equation 13-16 summarizes this in a more precise way:

$$\begin{array}{ll} \text{If } I_{NW} \le 1{,}300: & LC_{NW} = LC_{NW1} \\ \text{If } I_{NW} \ge 1{,}950: & LC_{NW} = LC_{NW2} \\ \text{If } 1{,}300 < I_{NW} < 1{,}950: & LC_{NW} = LC_{NW3} \\ \text{If } LC_{NW1} \ge LC_{NW2}: & LC_{NW} = LC_{NW2} \end{array}$$

Equation 13-16

Total Lane-Changing Rate

The total lane-changing rate LC_{ALL} of all vehicles in the weaving segment, in lane changes per hour, is computed from Equation 13-17:

$$LC_{ALL} = LC_W + LC_{NW}$$

Equation 13-17

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles in Weaving Segment

The heart of this methodology is the estimation of the average speeds of weaving and nonweaving vehicles in the weaving segment. These speeds are estimated separately because they are affected by different factors, and they can be significantly different from each other.

The speeds of weaving and nonweaving vehicles will be combined to find a space mean speed of all vehicles in the segment. This will then be converted to a density, which will determine the LOS.

Average Speed of Weaving Vehicles

The algorithm for predicting the average speed of weaving vehicles in a weaving segment may be generally stated as shown in Equation 13-18:

Equation 13-18

$$S_W = S_{MIN} + \left(\frac{S_{MAX} - S_{MIN}}{1 + W}\right)$$

where

S_W = average speed of weaving vehicles within the weaving segment (mi/h),

S_{MIN} = minimum average speed of weaving vehicles expected in a weaving segment (mi/h),

S_{MAX} = maximum average speed of weaving vehicles expected in a weaving segment (mi/h), and

W = weaving intensity factor (unitless).

The form of the model is logical and constrains the results to a reasonable range defined by the minimum and maximum speed expectations. The term $1 + W$ accommodates a weaving intensity factor that can be more or less than 1.0.

For this methodology, the minimum expected speed is taken to be 15 mi/h, and the maximum expected speed is the FFS, which may be modified to account for the impacts of inclement weather. At this time, there are no recommended procedures for adjusting the FFS to reflect incidents. As with all analyses, the FFS is best observed in the field, either on the subject facility or a similar facility. When it is measured, the FFS should be observed within the weaving segment.

In situations that require the FFS to be estimated, the model described in Chapter 12, Basic Freeway and Multilane Highway Segments, is used. The average speed of weaving vehicles within the weaving segment is estimated by using Equation 13-19 and Equation 13-20:

Equation 13-19

$$S_W = 15 + \left(\frac{FFS \times SAF - 15}{1 + W}\right)$$

Equation 13-20

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789}$$

where SAF is the speed adjustment factor (unitless). The speed adjustment factor can represent a combination of factors, including weather and work zone effects. Default speed adjustment factors and guidance for how to apply them are found in Chapter 11.

Note that weaving intensity is based on the total lane-changing rate within the weaving segment. More specifically, it is based on the hourly rate of lane changes per foot of weaving length. This might be thought of as a measure of the density of lane changes. In addition, the lane-changing rate itself depends on many demand and physical factors related to the design of the segment.

Average Speed of Nonweaving Vehicles

The average speed of nonweaving vehicles in a weaving segment is estimated by using Equation 13-21:

$$S_{NW} = FFS \times SAF - (0.0072LC_{MIN}) - \left(0.0048\frac{v}{N}\right)$$

Equation 13-21

Equation 13-21 treats nonweaving speed as a reduction from FFS. As would be expected, the speed is reduced as v/N increases. More interesting is the appearance of LC_{MIN} in the equation. LC_{MIN} is a measure of minimal weaving turbulence, assuming that weaving vehicles make only necessary lane changes. It depends on both the configuration of the weaving segment and weaving demand flow rates. Thus, nonweaving speeds decrease as weaving turbulence increases.

Average Speed of All Vehicles

The space mean speed of all vehicles in the weaving segment is computed by using Equation 13-22:

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W}\right) + \left(\frac{v_{NW}}{S_{NW}}\right)}$$

Equation 13-22

Step 8: Determine LOS

The average speed of all vehicles, computed in Step 7, must be converted to density by using Equation 13-23.

$$D = \frac{(v/N)}{S}$$

Equation 13-23

where D is density in passenger cars per mile per lane and all other variables are as previously defined. The density value obtained can then be used with Exhibit 13-6 to assign a LOS letter to the weaving segment.

LOS can be determined for weaving segments on freeways, multilane highways, and C-D roads.

4. EXTENSIONS TO THE METHODOLOGY

MULTIPLE WEAVING SEGMENTS

When a series of closely spaced merge and diverge areas creates overlapping weaving movements (between different merge–diverge pairs) that share the same segment of a roadway, a multiple weaving segment is created. In earlier editions of the HCM, a specific application of the weaving methodology for two-segment multiple weaving segments was included. While it was a logical extension of the methodology, it did not address cases in which three or more sets of weaving movements overlapped, nor was it well supported by field data.

Multiple weaving segments should be segregated into separate merge, diverge, and simple weaving segments, with each segment appropriately analyzed by using this chapter's methodology or that of Chapter 14, Freeway Merge and Diverge Segments. Chapter 12, Basic Freeway and Multilane Highway Segments, contains information relative to the process of identifying appropriate segments for analysis.

C-D ROADS

The methodology applies approximately to C-D roads, but its use may produce an overly negative view of operations.

A common design practice often results in weaving movements that occur on C-D roads that are part of a freeway interchange. The methodology of this chapter may be approximately applied to such segments. The FFS used must be appropriate to the C-D road. It would have to be measured on an existing or similar C-D road, since the predictive methodology of FFS given in Chapter 12 does not apply to such roads. Whether the LOS criteria of Exhibit 13-6 are appropriate is less clear. Many C-D roads operate at lower speeds and higher densities than do basic segments, and the criteria of Exhibit 13-6 may produce an inappropriately negative view of operations on a C-D road.

If the measured FFS of a C-D road is high (greater than or equal to 50 mi/h), reasonably accurate analysis results can be expected. At lower FFS values, results would be more approximate.

MULTILANE HIGHWAYS

Multilane highway weaving segments may be analyzed with this methodology, except in the vicinity of signalized intersections.

Weaving segments may occur on multilane highways. As long as such segments are a sufficient distance away from signalized intersections—so that platoon movements are not an issue—the methodology of this chapter may be approximately applied.

ML ACCESS SEGMENTS

Where managed lanes have defined intermittent access segments, two types of weaving movements may be created. Exhibit 13-12 illustrates the two types of situations.

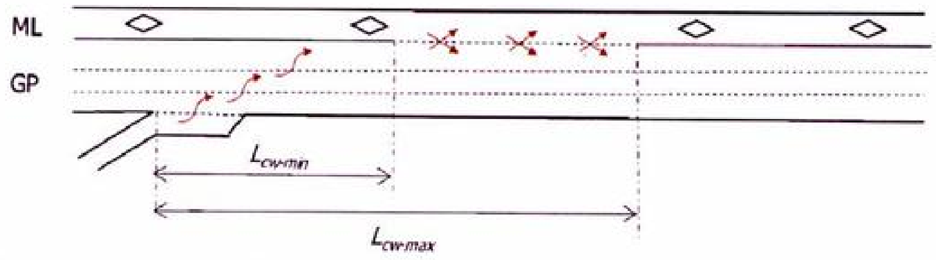


Exhibit 13-12
Weaving Movements Associated with Managed Lane Access and Egress

Note: ML = managed lane and GP = general purpose.

Exhibit 13-12 illustrates a managed lane with three general purpose freeway lanes. Where an on-ramp is near the ML access segment, on-ramp vehicles destined for the managed lane must cross all of the general purpose freeway lanes in the distance $L_{cw\text{-}min}$. The cross-weave demand can cause a reduction in the capacity of the general purpose lanes, which must be considered. While not shown, the same effect exists when an off-ramp is near the ML access segment, with the distance $L_{cw\text{-}min}$ measured from the end of the access segment to the off-ramp junction point.

The second type of weaving occurs within the ML access segment, as vehicles entering and exiting from the managed lane cross each other within the distance $L_{cw\text{-}max} - L_{cw\text{-}min}$. $L_{cw\text{-}min}$ is defined as the distance between the on-ramp gore area and the beginning of the ML access segment, while $L_{cw\text{-}max}$ is the distance from the gore to the end of the ML access segment.

Cross-Weaving Between Ramps and the ML Access Segment

The impact of cross-weaving movements on general purpose lane capacity is handled by using a CAF, as shown in Equation 13-24. The approach was developed as part of NCHRP Project 03-96 (*11*).

Equation 13-24

$$CAF = 1 - CRF$$

$$CRF = -0.0897 + 0.0252\ln(CW) - 0.00001453L_{cw\text{-}min} + 0.002967N_{GP}$$

where

CRF = capacity reduction factor (decimal),

CAF = capacity adjustment factor (decimal),

CW = cross-weave demand flow rate (pc/h),

$L_{cw\text{-}min}$ = cross-weave length (ft), and

N_{GP} = number of general purpose lanes (ln).

The capacity of the general purpose lanes is then computed as

Equation 13-25

$$c_{GPA} = c_{GP} \times CAF$$

where

c_{GPA} = adjusted capacity of the general purpose lanes (veh/h) and

c_{GP} = unadjusted capacity of the general purpose lanes, estimated by using basic freeway procedures in Chapter 12 (veh/h).

Weaving Within the ML Access Segment

Weaving within the ML access segment is treated by using the procedures of this chapter. The access segment is treated as a left-side ramp-weave segment with a length of $L_{cw\text{-}max} - L_{cw\text{-}min}$.

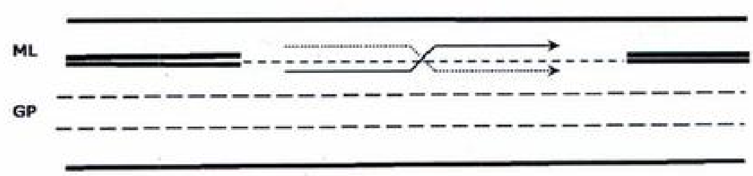
The interaction and weave turbulence effect is assumed to apply to the entire ML access segment, including all general purpose lanes. Consequently, the methodology is identical to the evaluation of a weaving segment on the left side of a freeway. When an ML access segment is evaluated as part of an extended freeway facility with managed lanes with the procedures in Chapter 10, the ML access segment represents the one exception where the general purpose and managed lanes are not treated as two separate lane groups. Instead, the calculated performance measures are applied across all lanes. In applying the weaving method, the basic segment capacity from Chapter 12, Basic Freeway and Multilane Highway Segments, should be used across all lanes when the weave capacity computations are performed (Equation 13-5).

Care should be taken when an overall managed lane facility is evaluated and the separation between the managed and general purpose lanes requires considering the adjacent friction effect, as described in Chapter 12. In those cases, the freeway facility methodology in Chapter 10 offers additional adjustments to the ML access segment for consistency with upstream or downstream ML basic segments.

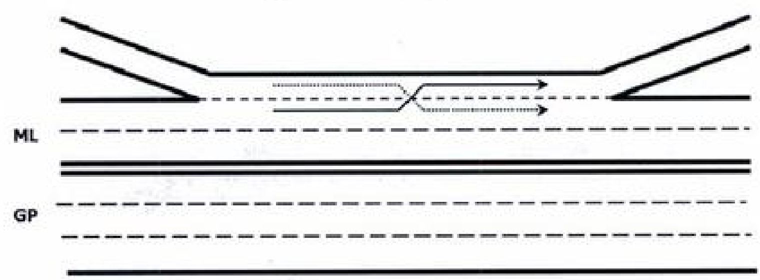
ML WEAVE SEGMENTS

The procedure described in this chapter may also be used to analyze an ML weave segment. An ML weave segment is limited to managed lane facilities with nontraversable separation from the general purpose lanes. The ML weave segment type is created when an on-ramp onto the managed lane is followed by an off-ramp from the managed lane and the two are connected by an auxiliary lane. The distinction between a ML weave and a ML access segment is illustrated in Exhibit 13-13.

Exhibit 13-13
Distinguishing ML Access and Weave Segments



(a) ML Access Segment



(b) ML Weave Segment

Note: ML = managed lane and GP = general purpose.

The procedure for analyzing an ML weave segment generally follows the methodology for a standard weaving segment. The only modification is the use of the managed lane's basic segment capacity from Chapter 12 in the weave capacity computations (Equation 13-5).

Care should be taken when an overall managed lane facility is evaluated, and the separation between the managed and general purpose lanes requires considering the adjacent friction effect, as described in Chapter 12. In those cases, the freeway facility methodology in Chapter 10 offers additional adjustments to the ML weave segment for consistency with upstream or downstream ML basic segments.

5. APPLICATIONS

This chapter's methodology is most often used to estimate the capacity and LOS of freeway weaving segments. The steps are most easily applied in the operational analysis mode, that is, all traffic and roadway conditions are specified, and a solution for the capacity (and *v/c* ratio) is found along with an expected LOS. However, other types of analysis are possible.

EXAMPLE PROBLEMS

Chapter 27, Freeway Weaving: Supplemental, contains seven detailed sample problems addressing the following scenarios:

1. LOS of a major weaving segment,
2. LOS of a ramp-weaving segment,
3. LOS of a two-sided weaving segment,
4. Design of a major weaving segment,
5. Construction of a service volume table for a weaving segment,
6. LOS of an ML access segment with cross weaving, and
7. ML access segment with a downstream off-ramp.

RELATED CONTENT IN THE HCMAG

The *Highway Capacity Manual Applications Guide* (HCMAG), accessible through the online HCM Volume 4, provides guidance on applying the HCM on freeway weaving segments. Case Study 4 goes through the process of identifying the goals, objectives, and analysis tools for investigating LOS on New York State Route 7, a 3-mi route north of Albany. The case study applies the analysis tools to assess the performance of the route, to identify areas that are deficient, and to investigate alternatives for correcting the deficiencies.

This case study includes the following problems related to freeway weaving segments:

1. Problem 2: Analysis of a complex interchange on the western end of the route
    a. Subproblem 2b: Weaving section LOS in the I-87/Alternate Route 7
2. Problem 3: Weaving and ramp analysis
    a. Subproblem 3a: Analysis of a freeway weaving section
    b. Subproblem 3c: Nonstandard ramp and weave analysis in the southwestern quadrant
    c. Subproblem 3d: Analysis of a C-D road

Other problems in the case study evaluate the operations of a freeway weaving segment as part of a greater freeway facility as discussed in the methodology in Chapter 10, Freeway Facilities Core Methodology.

Although the HCMAG was based on the HCM2000's procedures and chapter organization, the general process for applying the weaving procedure described in its case studies continues to be applicable to the methods in this chapter.

EXAMPLE RESULTS

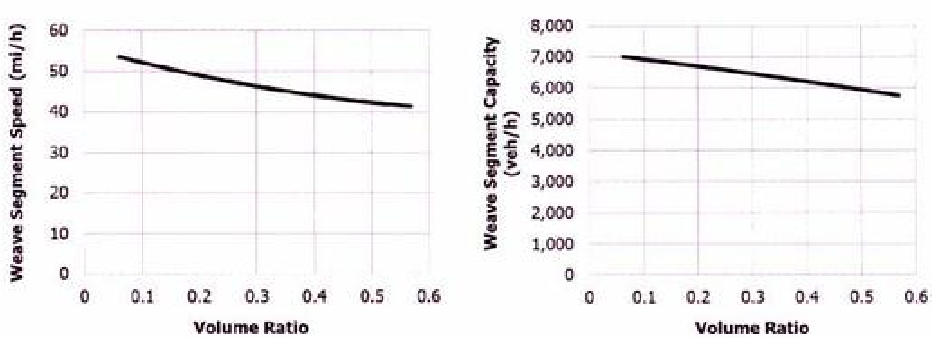
This section presents the results of applying this chapter's method in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs, as well as to help evaluate whether their analysis results are reasonable. The exhibits in this section are not intended to substitute for an actual analysis and are deliberately provided in a format large enough to depict general trends in the results but not large enough to pull out specific results.

Sensitivity of Results to Volume Ratio

Exhibit 13-14 presents illustrative results of the effect of volume ratio on the overall speed in the weaving segment, as well as on the weave segment capacity. Results are given for a standard ramp weave with $LC_{RF} = 1$, $LC_{FR} = 1$, and $N_{WL} = 2$. The analysis was performed by using a fixed total volume in the weaving segment and varying the proportion of weaving versus nonweaving traffic.

It can be seen that an increase in the volume ratio results in a reduction in weaving speed, due to increased turbulence in the segment. In addition, the segment capacity steadily decreases with an increase in volume ratio. The general trends in Exhibit 13-14 are expected to be similar for weaving segments with different geometric configurations.

Exhibit 13-14
Illustrative Effect of Volume Ratio on Weaving Speed and Capacity



(a) Weaving Segment Speed (b) Weaving Segment Capacity

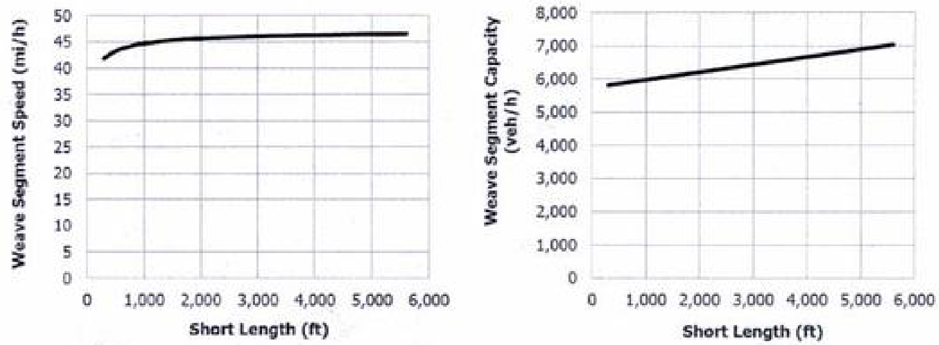
Note: Calculated by using this chapter's method, assuming short length L_S = 3,000 ft, $LC_{FR} = LC_{RF} = 1$, $LC_{RR} = 0$, $N_{WL} = 2$, FFS = 65 mi/h, interchange density = 0.8 interchanges/mi, PHF = 0.91, 3 lanes, $f_{HV} = 1$, and $V_{FF} + V_{RF} + V_{FR} + V_{RR}$ = 5,200 veh/h.

Sensitivity of Results to Segment Short Length

Exhibit 13-15 presents illustrative results of the effect of increasing the short length of the weaving segment on the weave segment speed and segment capacity. Results are given for a standard ramp weave with $LC_{RF} = 1$, $LC_{FR} = 1$, and $N_{WL} = 2$. The analysis used a fixed total volume and volume ratio.

The results show a linear increase in weave segment capacity with an increase in the segment short length. The results on weaving speed show lowest speed estimates for very short weaving segments, which increase as the short length increases. This increasing speed effect flattens for longer segment lengths.

Exhibit 13-15
Illustrative Effect of Short Length on Weaving Speed and Capacity



(a) Weaving Segment Speed

(b) Weaving Segment Capacity

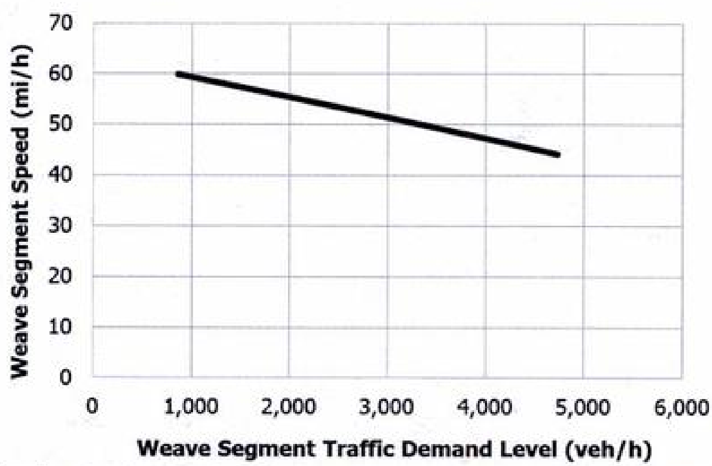
Note: Calculated by using this chapter's method, assuming LC_{FR} = LC_{RF} = 1, LC_{RR} = 0, N_W = 2, *FFS* = 65 mi/h, interchange density = 0.8 interchanges/mi, *PHF* = 0.91, 3 lanes, f_{HV} = 1, V_{FF} = 3,500 veh/h, V_{RF} = V_{FR} = 800 veh/h, and V_{RR} = 100 veh/h.

Sensitivity of Results to Weaving Segment Demand

Exhibit 13-16 presents illustrative results for an increase in weaving segment demand on the estimated segment speed. Results are given for a standard ramp weave with $LC_{RF} = 1$, $LC_{FR} = 1$, and $N_{WL} = 2$. Results are generated for a fixed proportion of weaving to nonweaving traffic by implementing a demand adjustment factor that proportionally increases all flows in the weaving segment.

Results suggest that an increase in demand will result in a steady decrease in the estimated speed in the weaving segment. Note that the capacity of the weaving segment is not affected in this experiment and is therefore fixed across the range of demands shown. An increase in demand therefore also corresponds to an increase in the demand-to-capacity ratio for the segment.

Exhibit 13-16
Illustrative Effect of Segment Demand on Weaving Speed



Note: Calculated by using this chapter's method, assuming short length (L_S) = 3,000 ft, LC_{FR} = LC_{RF}=1, LC_{RR} = 0, N_{WL} = 2, *FFS* = 65 mi/h, interchange density = 0.8 interchanges/mi, *PHF* = 0.91, 3 lanes, and f_{HV} = 1.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational, design, and planning and preliminary engineering.

Operational Analysis

The methodology of this chapter is most easily applied in the operational analysis mode. In this application, all weaving demands and geometric characteristics are known, and the output of the analysis is the expected LOS and the capacity of the segment. Secondary outputs include the average speed of component flows, the overall density in the segment, and measures of lane-changing activity.

Design Analysis

Design analysis is best accomplished by iterative operational analyses on a small number of candidate designs.

In design applications, the desired output is the length, width, and configuration of a weaving segment that will sustain a target LOS for given demand flows. This application is best accomplished by iterative operational analyses on a small number of candidate designs.

Generally, there is not a great deal of flexibility in establishing the length and width of a segment, and there is only limited flexibility in potential configurations. The location of intersecting facilities places logical limitations on the length of the weaving segment. The number of entry and exit lanes on ramps and the freeway itself limits the number of lanes to, at most, two choices. The entry and exit design of ramps and the freeway facility also produces a configuration that can generally only be altered by adding or subtracting a lane from an entry or exit roadway. Thus, iterative analyses of candidate designs are relatively easy to pursue, particularly with the use of HCM-replicating software.

Planning and Preliminary Engineering

Planning and preliminary engineering applications can have the same desired outputs as design applications: the geometric design of a weaving segment that can sustain a target LOS for specified demand flows. In addition, system performance monitoring applications may require planning-level applications of methodologies with simplified inputs. Further details and discussion on planning applications can be found in the *Planning and Preliminary Engineering Applications Guide to the HCM*.

In the planning and preliminary design phase, demand flows are sometimes stated as average annual daily traffic, in which case statistics must be converted to directional design hour volumes before this methodology is applied. Other planning applications use peak hour flow rates, which can be used directly in the methods in this chapter. A number of variables may be unknown (e.g., PHF and percentage of heavy vehicles), which may be replaced by default values.

Service Volumes and Service Flow Rates

The method can be applied to determine service volumes for LOS A–E for a specified set of conditions.

Service volume is the maximum hourly volume that can be accommodated without exceeding the limits of the various levels of service during the worst 15 min of the analysis hour. Service volumes can be found for LOS A–E. LOS F, which represents unstable flow, does not have a service volume.

Service flow rates are the maximum rates of flow (within a 15-min period) that can be accommodated without exceeding the limits of the various levels of service. As is the case for service volumes, service flow rates can be found for LOS A–E, but none is defined for LOS F. The relationship between a service volume and a service flow rate is as follows:

Equation 13-26

$$SV_i = SF_i \times PHF$$

where

SV_i = service volume for LOS *i* (pc/h),

SF_i = service flow rate for LOS *i* (pc/h), and

PHF = peak hour factor.

The methodology uses demand volumes in vehicles per hour converted to demand flow rates in passenger cars per hour. Therefore, service flow rates and service volumes would originally be estimated in terms of flow rates in passenger cars per hour. They would then be converted back to demand volumes in vehicles per hour.

Service volumes and service flow rates for weaving segments are stated in terms of the maximum volume (or flow) levels that can be accommodated without violating the definition of the LOS. The volume ratio, the proportion of total traffic that weaves, is held constant. Any change in the volume ratio would cause a change in all service volumes or service flow rates.

A large number of characteristics will influence service volumes and service flow rates, including the PHF, percent heavy vehicles, and any of the weaving segment's geometric attributes. Therefore, definition of a representative "typical" case with broadly applicable results is virtually impossible. Each case must be individually considered. An example is included in Chapter 27, Freeway Weaving: Supplemental, which is located in Volume 4.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for the application of alternative tools to the analysis of freeway weaving segments. Additional information on this topic, including supplemental example problems, may be found in Chapter 27, Freeway Weaving: Supplemental, located in Volume 4.

The limitations stated earlier in this chapter may be addressed by using available simulation tools. In some cases, the limitations are addressed by the Chapter 10 and 11 methodologies. The following conditions, which are beyond the scope of this chapter, are treated explicitly by simulation tools:

- *Ramp metering on entrance ramps forming part of the weaving segment.* These features are modeled explicitly by many tools.
- *Specific operating conditions when oversaturated conditions exist.* In this case, it is necessary to ensure that both the spatial and the temporal boundaries of the analysis extend beyond the congested operation.

- *Multiple weaving segments.* Multiple weaving segments were removed from this edition of the manual. They may be addressed to some extent by the procedures given in Chapters 10 and 11 for freeway facilities. Complex combinations of weaving segments may be analyzed more effectively by simulation tools, although such analyses might require extensive calibration of origin–destination characteristics.

Because of the interactions between adjacent freeway segments, alternative tools will find their principal application to freeways containing weaving segments at the facility level and not to isolated freeway weaving segments.

Additional Features and Performance Measures Available from Alternative Tools

In addition to offering more performance measures, alternative tools can identify specific point problems that could be overlooked in a segment-level analysis.

This chapter provides a methodology for estimating the speed and density in a weaving segment given traffic demands from both the weaving and the nonweaving movements. Capacity estimates and maximum weaving lengths are also produced. Alternative tools offer additional performance measures including delay, stops, queue lengths, fuel consumption, pollution, and operating costs.

As with most other procedural chapters in this manual, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of queuing caused by capacity constraints on the exit ramp of a weaving segment, including difficulty in making the required lane changes, is a good example of a situation that can benefit from the increased insight offered by a microscopic model. An example of the effect of exit ramp queue backup is presented in Chapter 27, Freeway Weaving: Supplemental.

Development of HCM-Compatible Performance Measures Using Alternative Tools

When alternative tools are used, the analyst must be careful to note the definitions of simulation outputs. The principal measures involved in the performance analysis of weaving segments are speed and delay. These terms are generally defined in the same manner by alternative tools; however, there are subtle differences among tools that often make it difficult to apply HCM criteria directly to the outputs of other tools. Performance measure comparisons are discussed in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Direct comparison of the numerical outputs from the HCM and alternative tools can be misleading.

Conceptual differences between the HCM and stochastic simulation models make direct comparison difficult for weaving segments. The HCM uses a set of deterministic equations developed and calibrated with field data. Simulation models treat each vehicle as a separate object to be propagated through the system. The physical and behavioral characteristics of drivers and vehicles in the HCM are represented in deterministic equations that compute passenger car equivalences, lane-changing rates, maximum weaving lengths, capacity, speed, and density. Simulation models apply the characteristics to each driver and vehicle, and these characteristics produce interactions between vehicles, the sum total of which determines the performance measures for a weaving segment.

One good example of the difference between microscopic and macroscopic modeling is how trucks are entered into the models. The HCM uses a conversion factor that increases the demand volumes to reflect the proportion of trucks. Simulation models deal with trucks explicitly by assigning more sluggish characteristics to each of them. The result is that HCM capacities, densities, and so forth are expressed in equivalent passenger car units, whereas the corresponding simulation values are represented by actual vehicles.

The HCM methodology estimates the speeds of weaving and nonweaving traffic streams, and on the basis of these estimates it determines the density within the weaving segment. Simulators that provide outputs on a link-by-link basis do not differentiate between weaving and nonweaving movements within a given link; thus, comparing these (intermediate) results with those of other tools would be somewhat difficult.

For a given set of inputs, simulation tools should produce answers that are similar to each other and to the HCM. Although most differences should be reconcilable through calibration and identification of point problems within a segment, precise numerical agreement is not generally a reasonable expectation.

Sample Calculations Illustrating Alternative Tool Applications

Supplemental computational examples illustrating the use of alternative tools are included in Chapter 27 of Volume 4.

Chapter 27, Freeway Weaving: Supplemental, contains three examples that illustrate the application of alternative tools to freeway weaving segments. All of the problems are based on Example Problem 1 presented in that chapter. Three questions are addressed by using a typical simulation tool:

1. Can the weaving segment capacity be estimated realistically by simulation by varying the demand volumes up to and beyond capacity?
2. How does the demand affect the performance in terms of speed and density in the weaving segment when the default model parameters are used for vehicle and behavioral characteristics?
3. How would the queue backup from a signal at the end of the off-ramp affect the weaving operation?

6. REFERENCES

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

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CHAPTER 14
FREEWAY MERGE AND DIVERGE SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

Freeway merge and diverge segments occur primarily at on-ramp and off-ramp junctions with the freeway mainline. They can also occur at major merge or diverge points where mainline roadways join or separate.

Freeway merge and diverge segments include ramp junctions and points where mainline roadways join or separate.

A ramp is a dedicated roadway providing a connection between two highway facilities. On freeways, all movements onto and off of the freeway are made at ramp junctions, which are designed to permit relatively high-speed merging and diverging maneuvers while limiting the disruption to the main traffic stream. Some ramps on freeways connect to collector–distributor (C-D) roadways, which in turn provide a junction with the freeway mainline. Ramps may appear on multilane highways, two-lane highways, arterials, and urban streets, but such facilities may also use signalized and unsignalized intersections at such junctions.

The procedures in this chapter focus on ramp–freeway junctions, but guidance is also provided to allow approximate use of such procedures on multilane highways and on C-D roadways.

This chapter provides guidance for using the procedures on multilane highways and C-D roadways.

CHAPTER ORGANIZATION

Chapter 14 presents methodologies for analyzing merge and diverge segment operations in uninterrupted-flow conditions. The chapter presents a methodology for evaluating isolated freeway merge and diverge segments, as well as several extensions to the core method, including analysis of two-lane ramps, left-hand ramps, and major merge and diverge segments.

Section 2 of this chapter presents the following concepts related to merge and diverge segments: overview and ramp components, classification of ramps, ramp and ramp junction analysis boundaries, ramp–freeway junction operations, base conditions, and level of service (LOS) criteria for merge and diverge segments.

Section 3 presents a method for evaluating automobile operations on merge and diverge segments. The method generates the following performance measures:

- Average speed of vehicles in the ramp influence area,
- Average density in the ramp influence area and in the aggregate across the entire segment, and
- LOS of the merge or diverge segment.

Section 4 extends the core method presented in Section 3 to incorporate considerations for single-lane ramp additions and lane drops, two-lane on-ramps and off-ramps, left-hand on-ramps and off-ramps, and ramp–freeway junctions on 10-lane freeways. The section also discusses extension of the method to major merge and diverge segments.

Section 5 presents guidance on using the results of a freeway merge or diverge segment analysis, including example results from the methods, information on the sensitivity of results to various inputs, and a discussion of service volume tables for merge and diverge segments.

RELATED HCM CONTENT

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

- Chapter 3, Modal Characteristics, where general characteristics of the motorized vehicle mode on freeway facilities are discussed;
- Chapter 4, Traffic Operations and Capacity Concepts, which provides background speed–flow–density concepts of freeway segments that form the basis of merge and diverge concepts presented in this chapter's Section 2;
- Chapter 10, Freeway Facilities Core Methodology, which provides a method for evaluating merge and diverge segments within an extended freeway facility and their interaction with basic segments and weaving segments;
- Chapter 11, Freeway Reliability Analysis, which provides a method for evaluating freeway facilities with weaving segments in a reliability context; the chapter also provides default speed and capacity adjustment factors that can be applied in this chapter's methodology;
- Chapter 12, Basic Freeway and Multilane Highway Segments, which must be used to evaluate a merge or diverge segment with a continuous lane add or drop, respectively;
- Chapter 28, Freeway Merges and Diverges: Supplemental, where additional methodological details and example problems for merge and diverge segments are presented;
- Case Study 4, New York State Route 7, in the *HCM Applications Guide* in Volume 4, which demonstrates how this chapter's methods can be applied to the evaluation of an actual freeway facility; and
- Section H, Freeway Analyses, in the *Planning and Preliminary Engineering Applications Guide to the HCM,* found in Volume 4, which describes how to incorporate this chapter's methods and performance measures into a planning effort.

2. CONCEPTS

OVERVIEW AND RAMP COMPONENTS

A ramp consists of three elements: the ramp roadway and two junctions. Junctions vary greatly in design and control features but generally fit into one of these categories:

- Ramp–freeway junctions (or a junction with a C-D roadway or multilane highway segment), or
- Ramp–street junctions.

When a ramp connects one freeway to another, the ramp consists of two ramp–freeway junctions and the ramp roadway. When a ramp connects a freeway to a surface facility, it generally consists of a ramp–freeway junction, the ramp roadway, and a ramp–street junction. A ramp connection to a surface facility (such as a multilane highway) or a C-D roadway that is designed for high-speed merging or diverging without control may be classified as a ramp–freeway junction for the purpose of analysis.

Ramps to multilane highways and C-D roadways that are designed for high-speed merging or diverging may be classified as ramp–freeway junctions for analysis purposes.

Ramp–street junctions may be uncontrolled, STOP-controlled, YIELD-controlled, or signalized. Analysis of ramp–street junctions is not detailed in this chapter; it is discussed in Chapter 23, Ramp Terminals and Alternative Intersections. Note, however, that an off-ramp–street junction, particularly if signalized, can result in queuing on the ramp roadway that can influence operations at the ramp–freeway junction and even mainline freeway conditions. Chapter 23 includes a methodology for estimating the queue storage ratio for the off-ramp approach; the queue is expected to spill back onto the freeway when this ratio exceeds 1.0. Mainline operations can also be affected by platoon entries created by ramp–street intersection control.

See Chapter 23 for a discussion of ramp–street junctions.

Ramp queuing from a junction of an off-ramp and street can influence the operations of the ramp–freeway junction and the upstream freeway.

The geometric characteristics of ramp–freeway junctions vary. The length and type (parallel, taper) of acceleration or deceleration lane(s), the free-flow speed (FFS) of both the ramp and the freeway in the vicinity of the ramp, the proximity of other ramps, and other elements all affect merging and diverging operations.

CLASSIFICATION OF RAMP SEGMENTS

Ramps and ramp–freeway junctions may occur in a wide variety of configurations. Some of the key characteristics of ramps and ramp junctions are summarized below:

- Ramp–freeway junctions that accommodate merging maneuvers are classified as *on-ramps*. Those that accommodate diverging maneuvers are classified as *off-ramps*. Where the junctions accommodate the merging of two major facilities, they are classified as *major merge* junctions. Where they accommodate the divergence of two major roadways, they are classified as *major diverge* junctions.
- The majority of ramps are right-hand ramps. However, some join with the left lane(s) of the freeway and are classified as left-hand ramps.

Left-hand ramps are considered as special cases in Section 4 of this chapter.

Merge and diverge segments with two lanes at the point of merge or diverge are considered as special cases in Section 4 of this chapter.

- Ramp roadways may have one or two lanes. At on-ramp freeway junctions, most two-lane ramp roadways merge into a single lane before merging with the freeway. In this case, the junction is classified as a one-lane ramp–freeway junction on the basis of the methodology of this chapter. In other cases, a two-lane ramp–freeway merge exists, and a special analysis model is used (see this chapter's Extensions to the Methodology section).
- For two-lane off-ramps, a single lane may exist at the ramp–freeway diverge, with the roadway widening to two lanes after the diverge. As with on-ramps, such cases are classified as one-lane ramp–freeway junctions on the basis of this chapter's methodology. However, two-lane off-ramp roadways often have two lanes at the diverge point as well. These are treated by using a special model (see this chapter's Extensions to the Methodology section).
- Ramp–freeway merge and diverge operations are affected by the number of lanes on the freeway segment (in one direction).
- Ramp–freeway merge and diverge operations may be affected by the proximity of adjacent ramps and the demand flow rates on those ramps.

The number of combinations of these characteristics that can occur is large. For any analysis, all of these (and other) characteristics must be specified if meaningful results are to be obtained.

RAMP AND RAMP JUNCTION ANALYSIS BOUNDARIES

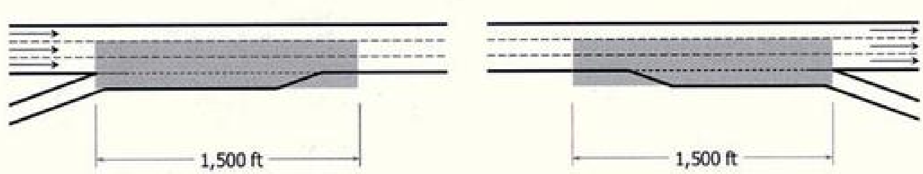
With undersaturated conditions, the operational impacts of ramp–freeway junctions occur within a 1,500-ft-long influence area.

Ramps and ramp junctions do not operate independently of the roadways they connect. Thus, operating conditions on the main roadways can affect operations on the ramp and ramp junctions, and vice versa. In particular, a breakdown (LOS F) at a ramp–freeway junction may have serious effects on the freeway upstream or downstream of the junction. Freeway operations can be affected for miles in the worst cases.

The influence area includes the acceleration/deceleration lane and the right two lanes of the freeway (left two lanes for left-hand ramps).

However, for most stable operations, studies (*1*) have shown that the operational impacts of ramp–freeway junctions are more localized. Thus, the methodology presented in this chapter predicts the operating characteristics within a defined ramp influence area. For right-hand on-ramps, the ramp influence area includes the acceleration lane(s) and Lanes 1 and 2 of the freeway mainline (rightmost and second rightmost) for a distance of 1,500 ft downstream of the merge point. For right-hand off-ramps, the ramp influence area includes the deceleration lane(s) and Lanes 1 and 2 of the freeway for a distance of 1,500 ft upstream of the diverge point. Exhibit 14-1 illustrates the definition of ramp influence areas. For left-hand ramps, the two leftmost lanes of the freeway are affected.

Exhibit 14-1
Ramp Influence Areas Illustrated



(a) Merge Influence Area (b) Diverge Influence Area

Where ramp or weaving influence areas overlap, the worst LOS of the overlapping areas is applied.

In many cases, the influence areas of adjacent ramps may overlap one another. In such cases, each influence area is analyzed separately with the methodology of this chapter. For the overlap area, the analysis resulting in the worse operating characteristics or LOS is applied. This general approach also applies to merge or diverge influence areas that overlap weaving segments.

RAMP–FREEWAY JUNCTION OPERATIONAL CONDITIONS

Ramp–freeway junctions create turbulence in the merging or diverging traffic stream. In general, the turbulence is the result of high lane-changing rates.

Ramp influence areas experience higher rates of lane-changing than normally occur in basic freeway segments.

The action of individual merging vehicles entering the Lane 1 traffic stream creates turbulence in the vicinity of the ramp. Approaching freeway vehicles move toward the left to avoid the turbulence. Thus, the ramp influence area experiences a higher rate of lane-changing than is normally present on ramp-free portions of freeway.

At off-ramps, the basic maneuver is a diverge, which is a single traffic stream separating into two streams. Exiting vehicles must occupy the lane(s) adjacent to the off-ramp (Lane 1 for a single-lane right-hand off-ramp). Thus, as the off-ramp is approached, vehicles leaving the freeway must move to the right. This causes other freeway vehicles to redistribute as they move left to avoid the turbulence of the immediate diverge area. Again, the ramp influence area has a higher rate of lane-changing than is normally present on ramp-free portions of freeway.

Vehicle interactions are dynamic in ramp influence areas. Approaching freeway through vehicles will move left as long as there is capacity to do so. Whereas the intensity of ramp flow influences the behavior of through freeway vehicles, general freeway congestion can also limit ramp flow and cause diversion to other interchanges or routes.

Exhibit 14-1 and the accompanying discussion relate to single-lane right-hand ramps. For two-lane right-hand ramps, the characteristics are basically the same, except that two acceleration or deceleration lanes may be present. For left-hand ramps, merging and diverging obviously take place on the left side of the freeway. This chapter's methodology is based on right-hand ramps, but modifications allowing the adaptation of the methodology to consider left-hand ramps are presented in the Extensions to the Methodology section.

BASE CONDITIONS

Base conditions for merge and diverge segments are the same as for other types of freeway segments.

The base conditions for the methodology presented in this chapter are the same as for other types of freeway segments:

- No heavy vehicles,
- 12-ft lanes,
- Adequate lateral clearances (≥6 ft), and
- Motorists who are familiar with the facility.

CAPACITY OF MERGE AND DIVERGE SEGMENTS

The base capacity of merge and diverge segments is the same as the corresponding capacity of a basic segment, which in turn is initially a function of the segment FFS as described in Chapter 12, Basic Freeway and Multilane Highway Segments. These base capacities reflect ideal conditions on a facility before consideration of any capacity-reducing effects. For example, the base capacities assume no heavy vehicles; no grades; and no additional friction effects due to poor pavement conditions, narrow lanes, or lighting conditions. The base capacities further do not include the effects of nonrecurring sources of congestion, such as severe weather, incidents, or work zones. Therefore, calibration of the base capacity to reflect local conditions may be necessary, especially when a segment is evaluated in the context of an extended freeway facility.

In the case of merge areas (and to a lesser extent diverge areas), some research has pointed out that the capacity can be reduced further as a result of the merge turbulence generated when a segment has both heavy mainline and heavy on-ramp flow. A merge segment with low on-ramp traffic (and thus little resulting merge turbulence) is expected to have a capacity similar to that of a basic segment, but some merge segments that function as active bottlenecks may have capacities below that of a basic segment.

While no national model exists for estimating the capacity of a merge or diverge segment as a function of on-ramp demand, mainline demand, lane configuration, acceleration/deceleration lane length, and so forth, several sources in the literature suggest that the resulting capacities can be less than those of a basic segment, as shown in Exhibit 14-2. The values in the exhibit are from a study of metered on-ramps, and capacities of unmetered sites may be different. Note that capacity is related to the "maximum prebreakdown flow" shown in Exhibit 14-2. The values are given in vehicles per hour per lane and would be higher if converted to passenger cars per hour per lane on the basis of truck presence. Chapter 12 offers additional discussion of prebreakdown capacity and the queue discharge flow rate.

Exhibit 14-2
Capacity Estimates at Merge Bottleneck Locations (veh/h/ln)

		Average (Standard Deviation)		
Location	**No. of Lanes**	**Breakdown Flow**	**Maximum Prebreakdown Flow**	**Queue Discharge Flow**
Minneapolis, Minn.	2	1,876 (218)	2,181 (163)	1,644 (96)
Portland, Ore.	2	2,010 (246)	2,238 (161)	1,741 (146)
Toronto, Canada	3	2,090 (247)	2,330 (162)	1,865 (124)
Sacramento, Calif.	3	1,943 (199)	2,174 (107)	1,563 (142)
Sacramento, Calif.	4	1,750 (256)	2,018 (108)	1,567(115)
San Diego, Calif.	4	1,868 (160)	2,075 (113)	1,665 (85)
San Diego, Calif.	5	1,774 (160)	1,928 (70)	1,635 (66)

Source: Elefteriadou (*2*).

The analyst should consider these values in estimating the merge segment capacity in the presence of high on-ramp and freeway flows and should validate through local data whenever possible. A correct calibration of the merge and diverge segment capacity is especially important in the context of a freeway facilities analysis in Chapter 10, Freeway Facilities Core Methodology. Reduced capacity values can be implemented in the merge/diverge methodology through the use of a capacity adjustment factor (CAF), as described in Section 3 of the chapter.

LOS CRITERIA FOR MERGE AND DIVERGE SEGMENTS

LOS A–E are defined in terms of density; LOS F exists when demand exceeds capacity.

Merge/diverge segment LOS is defined in terms of density for all cases of stable operation (LOS A–E). LOS F exists when the freeway demand exceeds the capacity of the upstream (diverges) or downstream (merges) freeway segment or when the on- or off-ramp demand exceeds the on- or off-ramp capacity.

At LOS A, unrestricted operations exist, and the density is low enough to permit smooth merging or diverging with little turbulence in the traffic stream. At LOS B, merging and diverging maneuvers become noticeable to through drivers, and minimal turbulence occurs. At LOS C, speed within the ramp influence area begins to decline as turbulence levels become much more noticeable. Both ramp and freeway vehicles begin to adjust their speeds to accomplish smooth transitions. At LOS D, turbulence levels in the influence area become intrusive, and virtually all vehicles slow to accommodate merging or diverging maneuvers. Some ramp queues may form at heavily used on-ramps, but freeway operation remains stable. LOS E represents operating conditions approaching or at capacity. Small changes in demand or disruptions within the traffic stream can cause both ramp and freeway queues to form.

LOS F defines operating conditions within queues that form on both the ramp and the freeway mainline when capacity is exceeded by demand. For on-ramps, LOS F exists when the total demand flow rate from the upstream freeway segment and the on-ramp exceeds the capacity of the downstream freeway segment. For off-ramps, LOS F exists when the total demand flow rate on the approaching upstream freeway segment exceeds the capacity of the upstream freeway segment. LOS F also occurs when the off-ramp demand exceeds the capacity of the off-ramp. When on-ramp demand exceeds on-ramp capacity, the ramp demand reaching the merge area is limited to the capacity of the on-ramp. Queues will develop at the entry to the ramp, but the merge area may experience stable operations.

Exhibit 14-3 summarizes the LOS criteria for freeway merge and diverge segments. These criteria apply to all ramp–freeway junctions and may also be applied to major merges and diverges; high-speed, uncontrolled merge or diverge ramps on multilane highway sections; and merges and diverges on freeway C-D roadways. LOS is not defined for ramp roadways, while the LOS of a ramp–street junction is defined in Chapter 23, Ramp Terminals and Alternative Intersections.

Exhibit 14-3
LOS Criteria for Freeway Merge and Diverge Segments

LOS	Density (pc/mi/ln)
A	≤10
B	>10–20
C	>20–28
D	>28–35
E	>35
F	Demand exceeds capacity

3. CORE METHODOLOGY

SCOPE OF THE METHODOLOGY

This chapter focuses on the operation of ramp–freeway junctions. The procedures may be applied in an approximate manner to completely uncontrolled ramp terminals on other types of facilities, such as multilane highways, two-lane highways, and freeway C-D roadways that are part of interchanges.

This chapter's procedures can be used to identify likely congestion at ramp–freeway junctions and to analyze undersaturated operations at ramp–freeway junctions. Chapter 10, Freeway Facilities Core Methodology, provides procedures for a more detailed analysis of oversaturated flow and congested conditions along a freeway section, including weaving, merge and diverge, and basic freeway segments.

The procedures in this chapter result primarily from studies conducted under National Cooperative Highway Research Program Project 03-37 (*1, 3*). Some special applications resulted from adaptations of procedures developed in the 1970s (*4*). American Association of State Highway and Transportation Officials policies (*5*) contain additional material on the geometric design and design criteria for ramps.

Spatial and Temporal Limits

As discussed, the methodology of this chapter focuses on the defined ramp influence area for each merge and diverge segment (Exhibit 14-1). The influence area is generally limited to the two rightmost freeway lanes and any acceleration or deceleration lanes present, for a distance of 1,500 ft downstream of the merge point or upstream of the diverge point. Where LOS F is experienced, queues can extend this influence for much greater distances. Such cases must be analyzed by using the procedures of Chapters 10 and 11 on freeway facilities.

Performance Measures

The methodology of this chapter results in predictions of the average speed and density of vehicles within the ramp influence area. It also provides estimates of the speeds and densities of lanes not included in the ramp influence area (which apply along the 1,500-ft length of the influence area) and estimates of average speeds and densities for all lanes of the freeway.

Strengths of the Methodology

This chapter's procedures were developed on the basis of extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent an expert consensus. Simulation packages generally do not relate geometric design details with operational performance the way it is done in this method.

The HCM procedure's strengths are as follows:

- The methodology provides capacity estimates. Simulators do not provide capacity estimates directly; they can be obtained by devising a data collection scheme in the simulator. Furthermore, the user can modify those simulated capacities by modifying specific input values, such as the minimum acceptable headway.
- The methodology explicitly considers the impacts of the presence of upstream and downstream ramps, as well as their respective demands.
- It produces a single deterministic estimate of density, which is important for some purposes, such as development impact review.

Limitations of the Methodology

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

- Special lanes, such as high-occupancy vehicle (HOV) lanes, as ramp entry lanes;
- Ramp metering; or
- Intelligent transportation system features.

The methodology does not explicitly take into account posted speed limits or level of police enforcement. In some cases, low speed limits and strict enforcement could result in lower speeds and higher densities than those anticipated by this methodology.

Alternative Tool Considerations

Merging and diverging segments can be analyzed with a variety of stochastic and deterministic simulation tools that address freeways. These tools can be useful in analyzing the extent of congestion when there are failures within the simulated facility range and when interaction with other freeway segments and facilities is present.

REQUIRED DATA AND SOURCES

The analysis of a ramp–freeway junction requires details concerning the junction under analysis and adjacent upstream and downstream ramps, in addition to the data required for a typical freeway analysis.

Exhibit 14-4 lists the information necessary for applying the freeway merge and diverge segment methodology and suggests potential sources for obtaining these data. It suggests default values for use when segment-specific information is not available. The user is cautioned that every use of a default value instead of a field-measured, segment-specific value may make the analysis results more approximate and less related to the conditions that describe the highway. HCM defaults should only be used when (*a*) field data cannot be collected and (*b*) locally derived defaults do not exist.

Exhibit 14-4
Required Input Data, Potential Data Sources, and Default Values for Freeway Merge and Diverge Segment Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Number of lanes	Road inventory, aerial photo	Must be provided
Ramp type	Road inventory, aerial photo	Must be provided
Number of lanes on ramp	Road inventory, aerial photo	1
Ramp location (right, left)	Road inventory, aerial photo	Right side
Length of acceleration lane	Road inventory, aerial photo	800 ft
Length of deceleration lane	Road inventory, aerial photo	400 ft
Presence of upstream and downstream ramps	Road inventory, aerial photo	None, isolated ramp
Terrain type (level, rolling, specific grade)	Design plans, analyst judgment	Must be provided
Free-flow speed (mi/h)	Direct speed measurements, estimate from design speed or speed limit	Speed limit + 5 mi/h
Ramp free-flow speed (mi/h)	Direct speed measurements, estimate from design speed or speed limit	35 mi/h
	Demand Data	
Hourly demand volume on freeway (veh/h)	Field data, modeling	Must be provided
Hourly demand volume on ramp (veh/h)	Field data, modeling	Must be provided
Hourly demand volume on upstream or downstream ramp (veh/h)	Field data, modeling	None, isolated ramp
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor[a] (decimal)	Field data	0.94 urban and rural
Speed and capacity adjustment factors for driver population[b]	Field data	1.0
Speed and capacity adjustment factors for weather and incidents[c]	Field data	1.0
Heavy vehicle percentage (%)	Field data	5% urban, 12% rural[d]

Notes: ***Bold italic*** indicates high sensitivity (>20% change) of service measure to the choice of default value.
Bold indicates moderate sensitivity (10%–20% change) of service measure to the choice of default value.
[a] Moderate to high sensitivity of service measures for very low PHF values. See the discussion in the text. PHF is not required when peak 15-min demand volumes are provided.
[b] See Chapter 26 in Volume 4 for default adjustment factors for driver population.
[c] See Chapter 11 for default capacity and speed adjustment factors for weather and incidents.
[d] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.

The exhibit distinguishes between urban and rural conditions for certain defaults. The classification of a facility as urban or rural is made on the basis of the Federal Highway Administration smoothed or adjusted urbanized boundary definition (*6*), which in turn is derived from Census data.

Care should be taken in using default values. The service measures are sensitive to some of the input data listed in Exhibit 14-4. For example, the FFS and the length of the acceleration lane can result in a 10%–20% change in the service measure when they are varied over their normal range. A very low peak hour factor (PHF) value (0.60) can bring about a greater than 20% change, compared with the results obtained for the default value for PHF; more typical PHFs vary the service measure results by less than 10%. Traffic demand volumes on mainline and ramp can change the output by more than 20%, and changes in heavy vehicle percentages can result in a 10%–20% change in the service measure. Other inputs change the service measure result by less than 10% when they are varied over their normal range.

Data Describing the Freeway

The following information concerning the freeway mainline is needed to conduct an analysis:

1. FFS: 55–75 mi/h;
2. Number of mainline freeway lanes: 2–5;
3. Terrain: level or rolling, or percent grade and length;
4. Heavy vehicle presence: percent trucks and buses;
5. Demand flow rate immediately upstream of the ramp–freeway junction;
6. PHF: up to 1.00; and
7. Driver population speed and capacity adjustment factors: defaults to 1.00 (see Chapter 26 for additional guidance)

FFS is best measured in the field but can be estimated by using the methodology for basic freeway segments or multilane highways, as applicable.

The freeway FFS is best measured in the field. If a field measurement is not available, FFS may be estimated by using the methodology for basic freeway segments presented in Chapter 12, Basic Freeway and Multilane Highway Segments. To use this methodology, information on lane widths, lateral clearances, number of lanes, and total ramp density is required. If the ramp junction is located on a multilane highway or C-D roadway, the FFS range is somewhat lower (45–60 mi/h) and can be estimated by using the methodology in Chapter 12 if no field measurements are available. The methodology can be applied to facilities with any FFS. Its use with multilane highways or C-D roadways must be considered approximate, however, since it was not calibrated with data from these types of facilities.

Where the ramp–freeway junction is on a specific grade, the length of the grade is measured from its beginning to the point of the ramp junction.

The driver population speed and capacity adjustment factors are generally set to 1.00 unless the traffic stream consists primarily of drivers who are not regular users of the facility. In such cases, an appropriate value should be based on field observations at the location under study or at similar nearby locations. Additional guidance on these factors is provided in Chapter 26.

Data Describing the Ramp–Freeway Junction

The following information concerning the ramp–freeway junction is needed to conduct an analysis:

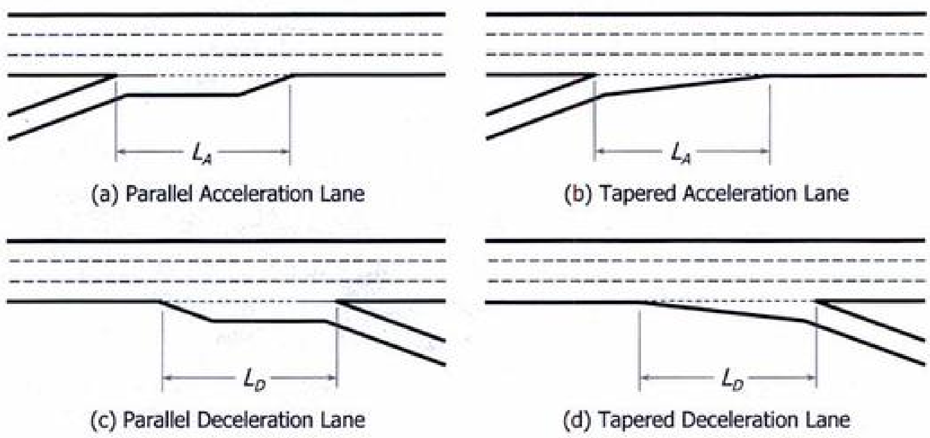
1. Type of ramp–freeway junction: merge, diverge, major merge, major diverge;
2. Side of junction: right-hand, left-hand;
3. Number of lanes on ramp roadway: 1 lane, 2 lanes;
4. Number of ramp lanes at ramp–freeway junction: 1 lane, 2 lanes;
5. Length of acceleration/deceleration lane(s);
6. FFS of the ramp roadway: 20–50 mi/h;
7. Ramp terrain: level, rolling, or mountainous; or percent grade, length;
8. Demand flow rate on ramp;

9. Heavy vehicle presence: percent trucks and buses;
10. PHF: up to 1.0;
11. Driver population speed and capacity adjustment factors: up to 1.0; and
12. For adjacent upstream or downstream ramps,
    a. Upstream or downstream distance to the merge or diverge under study,
    b. Demand flow rate on the upstream or downstream ramp, and
    c. PHF and heavy vehicle percentages for the upstream or downstream ramp.

The length of the acceleration or deceleration lane includes the tapered portion of the ramp.

The length of the acceleration or deceleration lane includes the tapered portion of the ramp. Exhibit 14-5 illustrates lengths for both parallel and tapered ramp designs.

Exhibit 14-5
Measuring the Length of Acceleration and Deceleration Lanes



Source: Roess et al. (*3*).

Length of Analysis Period

The analysis period for any freeway analysis, including ramp junctions, is generally the peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

OVERVIEW OF THE METHODOLOGY

Exhibit 14-6 illustrates the computational methodology applied to the analysis of ramp–freeway junctions. The analysis is generally entered with known geometric and demand factors. The primary outputs of the analysis are LOS and capacity. The methodology estimates the density and speed in the ramp influence area, as well as capacities, speeds, and densities for the entire segment across all lanes.

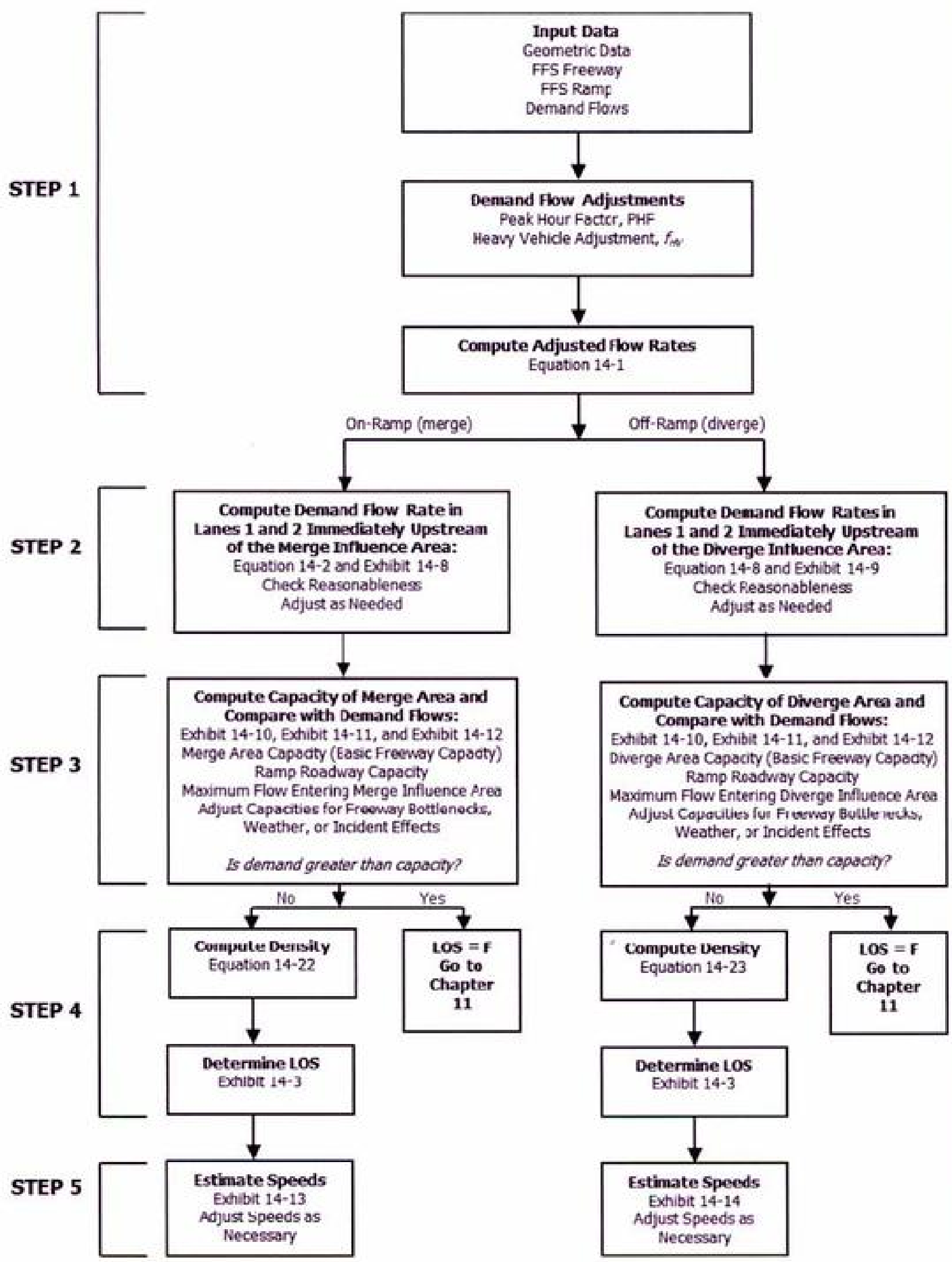
The computational process illustrated in Exhibit 14-6 may be categorized into five primary steps:

1. Specifying input variables and converting demand volumes to demand flow rates in passenger cars per hour under equivalent base conditions;

2. Estimating the flow remaining in Lanes 1 and 2 of the freeway immediately upstream of the merge or diverge influence area;
3. Estimating the capacity of the merge or diverge area and comparing the capacity with the converted demand flow rates;
4. For stable operations (i.e., demand is less than or equal to capacity), estimating the density within the ramp influence area and determining the expected LOS; and
5. When desired, estimating the average speed of vehicles within the ramp influence area.

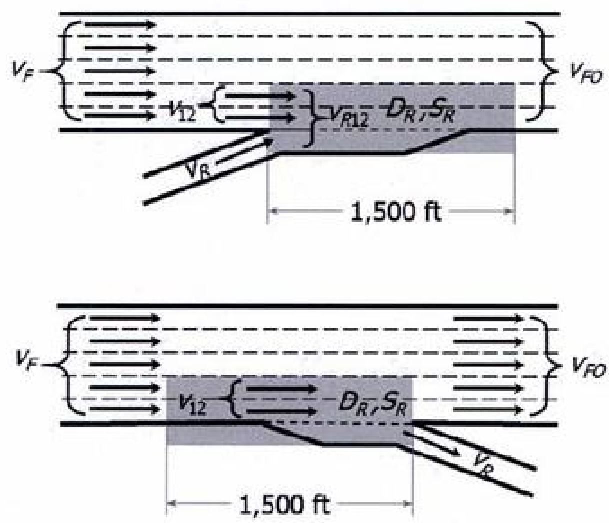
Each step is discussed in detail in the sections that follow.

Exhibit 14-6
Flowchart for Analysis of Ramp–Freeway Junctions



As previously discussed, the methodology focuses on modeling the operating conditions within the ramp influence area, as defined in Exhibit 14-1. Because the ramp influence area includes only Lanes 1 and 2 of the freeway, an important part of the methodology involves predicting the number of approaching freeway vehicles that remain in these lanes immediately upstream of the ramp–freeway junction. While operations in other freeway lanes may be affected by merging and diverging maneuvers, particularly under heavy flow, the defined influence area experiences most of the operational impacts across all levels of service (except LOS F). At breakdown, queues and operational impacts may extend well beyond the defined influence area. Exhibit 14-7 illustrates key variables involved in the methodology.

Exhibit 14-7
Key Ramp Junction Variables



The variables illustrated in Exhibit 14-7 are defined as follows:

v_F = flow rate on freeway immediately upstream of the ramp influence area under study (pc/h),

v_{12} = flow rate in freeway Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h),

v_{FO} = flow rate on the freeway immediately downstream of the merge or diverge area (pc/h),

v_R = flow rate on the on-ramp or off-ramp (pc/h),

v_{R12} = sum of the flow rates in Lanes 1 and 2 and the ramp flow rate (on-ramps only) (pc/h),

D_R = density in the ramp influence area (pc/mi/ln), and

S_R = average speed in the ramp influence area (mi/h).

COMPUTATIONAL STEPS

The methodology described in this section was calibrated for one-lane, right-side ramp–freeway junctions. All other cases—two-lane ramp junctions, left-side ramps, and major merge and diverge configurations—are analyzed with the modified procedures detailed in Section 4, Extensions to the Methodology.

The methodology was calibrated for one-lane, right-side ramp–freeway junctions. Other situations are addressed in the Extensions to the Methodology section.

Step 1: Specify Inputs and Convert Demand Volumes to Demand Flow Rates

All geometric and traffic variables for the ramp–freeway junction should be specified as inputs to the methodology, as discussed previously. Flow rates on the approaching freeway, on the ramp, and on any existing upstream or downstream adjacent ramps must be converted from hourly volumes (in vehicles per hour) to peak 15-min flow rates (in passenger cars per hour) under equivalent ideal conditions (Equation 14-1):

$$v_i = \frac{V_i}{PHF \times f_{HV}}$$

Equation 14-1

where

v_i = demand flow rate for movement i (pc/h),

V_i = demand volume for movement i (veh/h),

PHF = peak hour factor (decimal), and

f_{HV} = adjustment factor for heavy vehicle presence (decimal).

If demand data or forecasts are already stated as 15-min flow rates, *PHF* is set at 1.00. Adjustment factors are the same as those used in Chapter 12, Basic Freeway and Multilane Highway Segments. These can also be used when the primary facility is a multilane highway or a C-D roadway in a freeway interchange.

Step 2: Estimate the Approaching Flow Rate in Lanes 1 and 2 of the Freeway Immediately Upstream of the Ramp Influence Area

Because the ramp influence area includes Lanes 1 and 2 of the freeway (for a right-hand ramp), a critical step in the analysis is estimating the total flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area.

The distribution of freeway vehicles approaching a ramp influence area is affected by a number of variables:

- Total freeway flow approaching the ramp influence area v_F (pc/h),
- Total on- or off-ramp flow v_R (pc/h),
- Total length of the acceleration lane L_A or deceleration lane L_D (ft), and
- FFS of the ramp at the junction point S_{FR} (mi/h).

The lane distribution of approaching freeway vehicles may also be affected by adjacent upstream or downstream ramps. Nearby ramps can influence lane distribution as drivers execute lane changes to position themselves for ramp movements at adjacent ramps. For example, an on-ramp located only a few hundred feet upstream of a subject ramp may result in additional vehicles in

Lanes 1 and 2 at the subject ramp. A downstream off-ramp near a subject ramp may contain additional vehicles in Lanes 1 and 2 destined for the downstream ramp.

Theoretically, the influence of adjacent upstream and downstream ramps does not depend on the cross section of the freeway. In practical terms, however, this methodology only accounts for such influences on six-lane freeways (three lanes in one direction). On four-lane freeways (two lanes in one direction), the determination of v_{12} is equal to the total entering volume on the freeway; since only Lanes 1 and 2 exist, all approaching freeway vehicles are, by definition, in Lanes 1 and 2 regardless of the proximity of adjacent ramps. On eight-lane (four lanes in one direction) or larger freeways, the data are insufficient to determine the impact of adjacent ramps on lane distribution.

In addition, two-lane ramps are never included in the consideration of "adjacent" ramps under these procedures. Similarly, ramps that form part of an adjacent weaving segment, or ramps that constitute lane additions or lane drops, should not be considered.

For six-lane freeways, the methodology includes a process for determining whether adjacent upstream and downstream ramps are close enough to influence lane distribution at a subject ramp junction. When such ramps are close enough, the following additional variables may be involved:

- Flow rate on the adjacent upstream ramp v_U (pc/h),
- Distance between the subject ramp junction and the adjacent upstream ramp junction L_{UP} (ft),
- Flow rate on the adjacent downstream ramp v_D (pc/h), and
- Distance between the subject ramp junction and the adjacent downstream ramp junction L_{DOWN} (ft).

The distance to adjacent ramps is measured between the points at which the left edge of the leftmost ramp lane meets the right-lane edge of the freeway.

In practical terms, the influence of adjacent ramps rarely extends more than approximately 8,000 ft. Nevertheless, whether an adjacent ramp on a six-lane freeway has influence should be determined by using the algorithms specified in this methodology.

Of all these variables, the total approaching freeway flow has the greatest impact on flow in Lanes 1 and 2. The models are structured to account for this phenomenon without distorting other relationships. Longer acceleration and deceleration lanes lessen turbulence as ramp vehicles enter or leave the freeway. This leads to lower densities and higher speeds in the ramp influence area. When the ramp has a higher FFS, vehicles can enter and leave the freeway at higher speeds, and approaching freeway vehicles tend to move left to avoid the possibility of high-speed turbulence. This produces greater presegregation and smoother flow across all freeway lanes.

While the models are similarly structured, there are distinct differences between the lane distribution impacts of on-ramps and off-ramps. Separate models are presented for each case in the sections that follow.

Estimating Flow in Lanes 1 and 2 for On-Ramps (Merge Areas)

The general model for on-ramps specifies that flow in Lanes 1 and 2 immediately upstream of the merge influence area is simply a proportion of the approaching freeway flow, as shown in Equation 14-2:

$$v_{12} = v_F \times P_{FM}$$

Equation 14-2

where

v_{12} = flow rate in Lanes 1 and 2 (pc/h),

v_F = total flow rate on freeway immediately upstream of the on-ramp (merge) influence area (pc/h), and

P_{FM} = proportion of freeway vehicles remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area (decimal).

Exhibit 14-8 shows the algorithms used to determine P_{FM} for on-ramps or merge areas. All variables in Exhibit 14-8 are as previously defined.

Three equations are provided for six-lane freeways. Equation 14-3 is the base case for isolated ramps and for cases in which adjacent ramps are not found to influence merging operations. Equation 14-4 addresses cases with an upstream adjacent off-ramp, and Equation 14-5 addresses cases with a downstream adjacent off-ramp. Adjacent on-ramps (either upstream or downstream) have not been found to have a statistically significant impact on operations and are therefore ignored; Equation 14-3 is applied in such cases.

Exhibit 14-8
Models for Predicting P_{FM} at On-Ramps or Merge Areas

No. of Freeway Lanes[a]	Model(s) for Determining P_{FM}	
4	$P_{FM} = 1.000$	
6	$P_{FM} = 0.5775 + 0.000028L_A$	**Equation 14-3**
	$P_{FM} = 0.7289 - 0.0000135(v_F + v_R) - 0.003296S_{FR} + 0.000063L_{UP}$	**Equation 14-4**
	$P_{FM} = 0.5487 + 0.2628(v_D/L_{DOWN})$	**Equation 14-5**
8	For $v_F/S_{FR} \leq 72$: $P_{FM} = 0.2178 - 0.000125v_R + 0.01115(L_A/S_{FR})$	
	For $v_F/S_{FR} > 72$: $P_{FM} = 0.2178 - 0.000125v_R$	

Selecting Equations for P_{FM} for Six-Lane Freeways

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	On	None	Equation 14-3
None	On	On	Equation 14-3
None	On	Off	Equation 14-5 or 14-3
On	On	None	Equation 14-3
Off	On	None	Equation 14-4 or 14-3
On	On	On	Equation 14-3
On	On	Off	Equation 14-5 or 14-3
Off	On	On	Equation 14-4 or 14-3
Off	On	Off	Equation 14-5 or 14-4 or 14-3

Notes: If an adjacent diverge on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 14-3.
[a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.

Adjacent upstream or downstream ramps do not affect the prediction of v_{12} for two-lane (one direction) freeway segments, since *all* vehicles are in Lanes 1 and 2. There are insufficient data to determine whether adjacent ramps influence lane distribution on four-lane (one direction) freeway segments, and thus no such impact is incorporated into this methodology.

Where an upstream or downstream adjacent off-ramp exists on a six-lane freeway, a determination as to whether the ramp is close enough to the subject merge area to influence the area's operation is necessary. The determination is made by finding the equilibrium separation distance L_{EQ}. If the actual distance is larger than or equal to L_{EQ}, Equation 14-3 should be used. If the actual distance is shorter than L_{EQ}, Equation 14-4 or Equation 14-5 should be used as appropriate. The equilibrium distance is obtained by finding the distance at which Equation 14-3 would yield the same value of P_{FM} as Equation 14-4 or Equation 14-5 as appropriate. This results in the following:

For adjacent upstream off-ramps, use Equation 14-6:

Equation 14-6

$$L_{EQ} = 0.214(v_F + v_R) + 0.444L_A + 52.32S_{FR} - 2{,}403$$

For adjacent downstream off-ramps, use Equation 14-7:

Equation 14-7

$$L_{EQ} = \frac{v_D}{0.1096 + 0.000107L_A}$$

where all terms are as previously defined.

When both adjacent upstream and downstream off-ramps are present, the larger resulting value of P_{FM} is used.

A special case exists when both an upstream and a downstream adjacent off-ramp are present. In such cases, two different values of P_{FM} could arise: one from consideration of the upstream ramp and the other from consideration of the downstream ramp (they cannot be considered simultaneously). In such cases, the analysis resulting in the larger value of P_{FM} is used.

When an adjacent off-ramp to a merge area on a six-lane freeway is not a one-lane, right-side off-ramp, apply Equation 14-3.

In addition, the algorithms used to include the impact of an upstream or downstream off-ramp on a six-lane freeway are only valid for single-lane, right-side adjacent ramps. Where adjacent off-ramps consist of two-lane junctions or major diverge configurations, where ramps are part of a lane add or weaving segment, or where they are on the left side of the freeway, Equation 14-3 is always applied, together with other modifications described in the Extensions to the Methodology section of this chapter.

Estimating Flow in Lanes 1 and 2 for Off-Ramps (Diverge Areas)

All off-ramp traffic approaching an off-ramp (diverge area) must be in freeway Lanes 1 and 2 immediately upstream of the ramp to execute the desired maneuver. Thus, for off-ramps, the flow in Lanes 1 and 2 consists of all off-ramp vehicles and a proportion of freeway through vehicles, as in Equation 14-8:

Equation 14-8

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

where

- v_{12} = flow rate in Lanes 1 and 2 of the freeway immediately upstream of the deceleration lane (pc/h),
- v_R = flow rate on the off-ramp (pc/h),
- v_F = flow rate on freeway immediately upstream of the ramp influence area under study (pc/h), and
- P_{FD} = proportion of through freeway traffic remaining in Lanes 1 and 2 immediately upstream of the deceleration lane (decimal).

For off-ramps, the point at which flows are defined is the beginning of the deceleration lane(s), regardless of whether this point is within or outside the ramp influence area.

Exhibit 14-9 contains the equations used to estimate P_{FD} at off-ramp diverge areas. As was the case for on-ramps (merge areas), the value of P_{FD} for four-lane freeways is fixed at 1.0, since only Lanes 1 and 2 exist.

Exhibit 14-9
Models for Predicting P_{FD} at Off-Ramps or Diverge Areas

No. of Freeway Lanes[a]	Model(s) for Determining P_{FD}		
4	$P_{FD} = 1.000$		
6	$P_{FD} = 0.760 - 0.000025v_F - 0.000046v_R$		Equation 14-9
	$P_{FD} = 0.717 - 0.000039v_F + 0.604(v_U/L_{UP})$	when $v_U/L_{UP} \le 0.2^b$	Equation 14-10
	$P_{FD} = 0.616 - 0.000021v_F + 0.124(v_D/L_{DOWN})$		Equation 14-11
8	$P_{FD} = 0.436$		

Selecting Equations for P_{FD} for Six-Lane Freeways

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	Off	None	Equation 14-9
None	Off	On	Equation 14-9
None	Off	Off	Equation 14-11 or Equation 14-9
On	Off	None	Equation 14-10 or Equation 14-9
Off	Off	None	Equation 14-9
On	Off	On	Equation 14-10 or Equation 14-9
On	Off	Off	Equation 14-11, Equation 14-10 or Equation 14-9
Off	Off	On	Equation 14-9
Off	Off	Off	Equation 14-11 or Equation 14-9

Notes: If an adjacent ramp on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 14-9.
[a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
[b] When $v_U/L_{UP} > 0.2$, use Equation 14-9.

For six-lane freeways, three equations are presented. Equation 14-9 is the base case for isolated ramps or for cases in which the impact of adjacent ramps can be ignored. Equation 14-10 addresses cases in which there is an adjacent upstream on-ramp, and Equation 14-11 addresses cases in which there is an adjacent downstream off-ramp. Adjacent upstream off-ramps and downstream on-ramps have not been found to have a statistically significant impact on diverge operations and may be ignored. All variables in Exhibit 14-9 are as previously defined.

Insufficient information is available to establish an impact of adjacent ramps on eight-lane freeways (four lanes in each direction). This methodology does not include such an impact.

Where an adjacent upstream on-ramp or downstream off-ramp on a six-lane freeway exists, a determination as to whether the ramp is close enough to the subject off-ramp to affect its operation is necessary. As was the case for on-ramps, this is done by finding the equilibrium distance L_{EQ}. This distance is determined when Equation 14-9 yields the same value of P_{FD} as Equation 14-10 (for adjacent upstream on-ramps) or Equation 14-11 (adjacent downstream off-ramps). When the actual distance between ramps is greater than or equal to L_{EQ}, Equation 14-9 is used. When the actual distance between ramps is less than L_{EQ}, Equation 14-10 or Equation 14-11 is used as appropriate.

For adjacent upstream on-ramps, use Equation 14-12 to find the equilibrium distance:

Equation 14-12

$$L_{EQ} = \frac{v_U}{0.071 + 0.000023v_F - 0.000076v_R}$$

For adjacent downstream off-ramps, use Equation 14-13:

Equation 14-13

$$L_{EQ} = \frac{v_D}{1.15 - 0.000032v_F - 0.000369v_R}$$

where all terms are as previously defined.

In cases where Equation 14-12 indicates that Equation 14-10 should be used to determine P_{FD}, but $v_U/L_{UP} > 0.20$, Equation 14-9 must be used as a default. This is due to the valid calibration range of Equation 14-10 and the fact that it will yield unreasonable results when v_U/L_{UP} exceeds 0.20. This will lead to step-function changes in P_{FD} for values just below or above $v_U/L_{UP} = 0.20$.

When both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present, the larger resulting value of P_{FD} is used.

In the special case that both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present, two solutions for P_{FD} may arise, depending on which adjacent ramp is considered (both ramps cannot be considered simultaneously). In such cases, the larger value of P_{FD} is used.

When an adjacent ramp to a diverge area on a six-lane freeway is not a one-lane, right-side ramp, apply Equation 14-9.

As was the case for merge areas, the algorithms used to include the impact of an upstream or downstream ramp on a six-lane freeway are only valid for single-lane, right-side adjacent ramps. Where adjacent ramps consist of two-lane junctions, major merge configurations, or major diverge configurations; where ramps are part of a lane drop or weaving segment; or where ramps are on the left side of the freeway, Equation 14-9 is always applied.

Checking the Reasonableness of the Lane Distribution Prediction

The algorithms of Exhibit 14-8 and Exhibit 14-9 were developed through regression analysis of a large database. Unfortunately, regression-based models may yield unreasonable or unexpected results when they are applied outside the strict limits of the calibration database, and they may have inconsistencies at their boundaries.

Reasonableness checks on the value of v_{12}.

Therefore, limits must be applied to predicted values of flow in Lanes 1 and 2 (v_{12}). The following limitations apply to all such predictions:

1. The average flow per lane in the outer lanes of the freeway (lanes other than 1 and 2) should not be higher than 2,700 pc/h/ln.
2. The average flow per lane in outer lanes should not be higher than 1.5 times the average flow in Lanes 1 and 2.

These limits guard against cases in which the predicted value of v_{12} implies an unreasonably high flow rate in outer lanes of the freeway. When either of these limits is violated, an adjusted value of v_{12} must be computed and used in the remainder of the methodology. These applications are discussed in the next two subsections.

Application to Six-Lane Freeways

On a six-lane freeway (three lanes in one direction), there is only one outer lane to consider. The flow rate in this outer lane (Lane 3) is given by Equation 14-14:

Equation 14-14

$$v_3 = v_F - v_{12}$$

where

v_3 = flow rate in Lane 3 of the freeway (pc/h/ln),

v_F = flow rate on freeway immediately upstream of the ramp influence area (pc/h), and

v_{12} = flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h).

Then, if v_3 is greater than 2,700 pc/h/ln, use Equation 14-15:

Equation 14-15

$$v_{12a} = v_F - 2{,}700$$

If v_3 is greater than $1.5 \times (v_{12}/2)$, use Equation 14-16:

Equation 14-16

$$v_{12a} = \left(\frac{v_F}{1.75}\right)$$

where v_{12a} equals the adjusted flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h) and all other variables are as previously defined.

In cases where both limitations on outer lane flow rate are violated, the result yielding the highest value of v_{12a} is used. The adjusted value replaces the original value of v_{12} and the analysis continues.

Application to Eight-Lane Freeways

On eight-lane freeways, there are two outer lanes (Lanes 3 and 4). Thus, the limiting values cited previously apply to the average flow rate per lane in these lanes. The average flow in these lanes is computed from Equation 14-17:

Equation 14-17

$$v_{av34} = \frac{v_F - v_{12}}{2}$$

where v_{av34} equals the flow rate in outer lanes (pc/h/ln) and all other variables are as previously defined.

Then, if v_{av34} is greater than 2,700, use Equation 14-18:

Equation 14-18

$$v_{12a} = v_F - 5{,}400$$

If v_{av34} is greater than $1.5 \times (v_{12}/2)$, use Equation 14-19:

Equation 14-19

$$v_{12a} = \left(\frac{v_F}{2.50}\right)$$

where all terms are as previously defined.

In cases where both limitations on outer lane flow rate are violated, the result yielding the highest value of v_{12a} is used. The adjusted value replaces the original value of v_{12} and the analysis continues.

Summary of Step 2

At this point, an appropriate value of v_{12} has been computed and adjusted as necessary.

Step 3: Estimate the Capacity of the Ramp–Freeway Junction and Compare with Demand Flow Rates

There are three major checkpoints for the capacity of a ramp–freeway junction:

Locations for checking the capacity of a ramp–freeway junction.

1. The capacity of the freeway immediately downstream of an on-ramp or immediately upstream of an off-ramp,
2. The capacity of the ramp roadway, and
3. The maximum flow rate entering the ramp influence area.

Freeway capacity immediately downstream of an on-ramp or upstream of an off-ramp is usually the controlling factor.

In most cases, the freeway capacity is the controlling factor. Studies (*1*) have shown that the turbulence in the vicinity of a ramp–freeway junction does not necessarily diminish the capacity of the freeway, especially when the entering volume from the on-ramp (or exiting traffic to the off-ramp) is low. However, other studies (*2*) have pointed to some merge and diverge segments having significantly lower capacities, with those segments acting as major bottlenecks along freeway facilities. With increasing turbulence in the merge area (and to a lesser extent, the diverge area), the segment capacity can be reduced, resulting in a breakdown of the segment and the overall freeway facility.

No national model exists for estimating the capacity of a merge or diverge segment as a function of on-ramp demand, mainline demand, lane configuration, or acceleration/deceleration lane length, although some estimates from the literature (*2*) were provided in Exhibit 14-2. In the absence of a national model, the analyst is encouraged to gather local data or rely on state or regional guidance to estimate the capacity of merge or diverge segments. The base capacity in this chapter can then be adjusted by using a capacity adjustment factor as described below.

The capacity of the ramp roadway is rarely a factor at on-ramps, but it can play a major role at off-ramp (diverge) junctions. Failure of a diverge junction is most often caused by a capacity deficiency on the off-ramp roadway or at its ramp–street terminal.

Failure of a diverge junction is usually caused by a capacity deficiency at the ramp–street terminal or on the off-ramp roadway.

While this methodology establishes a maximum desirable rate of flow entering the ramp influence area, exceeding this value does not cause a failure when other capacity values are not exceeded. Instead, it means that operations may be less desirable than indicated by the methodology. At off-ramps, the total flow rate entering the ramp influence area is merely the estimated value of v_{12}. However, at on-ramps, the on-ramp flow also enters the ramp influence area. Therefore, the total flow entering the ramp influence area at an on-ramp is given by Equation 14-20:

Equation 14-20

$$v_{R12} = v_{12} + v_R$$

where v_{R12} is the total flow rate entering the ramp influence area at an on-ramp (pc/h) and all other variables are as previously defined.

Exhibit 14-10 shows capacity values for ramp–freeway junctions. These are the same as the capacity of a basic freeway segment (Chapter 12) with the number of lanes entering or leaving the ramp junction. They are included here for convenience of use.

Exhibit 14-11 shows similar values for high-speed ramps on multilane highways and C-D roadways within freeway interchanges. Exhibit 14-12 shows the capacity of ramp roadways.

Exhibit 14-10
Capacity of Ramp–Freeway Junctions (pc/h)

FFS (mi/h)	Capacity of Upstream or Downstream Freeway Segment[a] Number of Lanes in One Direction				Maximum Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Maximum Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
	2	3	4	>4		
≥70	4,800	7,200	9,600	2,400/ln	4,600	4,400
65	4,700	7,050	9,400	2,350/ln	4,600	4,400
60	4,600	6,900	9,200	2,300/ln	4,600	4,400
55	4,500	6,750	9,000	2,250/ln	4,600	4,400

Notes: [a] Demand in excess of these capacities results in LOS F.
[b] Demand in excess of these values alone does not result in LOS F; operations may be worse than predicted by this methodology.

Exhibit 14-11
Capacity of High-Speed Ramp Junctions on Multilane Highways and C-D Roadways (pc/h)

FFS (mi/h)	Capacity of Upstream or Downstream Highway or C-D Segment[a] Number of Lanes in One Direction			Maximum Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Maximum Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
	2	3	>3		
≥60	4,400	6,600	2,200/ln	4,600	4,400
55	4,200	6,300	2,100/ln	4,600	4,400
50	4,000	6,000	2,000/ln	4,600	4,400
45	3,800	5,700	1,900/ln	4,600	4,400

Notes: [a] Demand in excess of these capacities results in LOS F.
[b] Demand in excess of these values alone does not result in LOS F; operations may be worse than predicted by this methodology.

Exhibit 14-12
Capacity of Ramp Roadways (pc/h)

Ramp FFS, S_{FR} (mi/h)	Single-Lane Ramps	Two-Lane Ramps
>50	2,200	4,400
>40–50	2,100	4,200
>30–40	2,000	4,000
≥20–30	1,900	3,800
<20	1,800	3,600

Notes: Capacity of a ramp roadway does not ensure an equal capacity at its freeway or other high-speed junction. Junction capacity must be checked against criteria in Exhibit 14-10 and Exhibit 14-11.

The two-lane ramp capacity in Exhibit 14-12 is based on limited data and thus may require local calibration. However, as noted above, the capacity of the actual merge or diverge junction typically controls the segment capacity, not the capacity of the ramp roadway itself.

Adjustments to Capacity for Bottlenecks, Inclement Weather, or Incidents

The capacity of basic freeway lanes, ramp roadways, or both may be adjusted further to account for high turbulence in the merge or diverge segment, as well as for the impacts of adverse weather, driver population, and traffic incidents. This adjustment is the same as that for other freeway segment types; default values are provided in Chapter 11, Freeway Reliability Analysis. The weather and incident adjustments are most commonly applied in the context of a reliability analysis as described in that chapter. For convenience, a brief summary is provided here.

The capacity of a merge or diverge segment is adjusted as follows:

Equation 14-21

$$c_{mda} = c_{md} \times CAF$$

where

c_{mda} = adjusted capacity of merge/diverge area (veh/h);

c_{md} = unadjusted capacity of merge/diverge area (veh/h); and

CAF = capacity adjustment factor, from Chapter 11 (unitless).

The CAF can have several components, including adjustments for merge or diverge turbulence, weather, incidents, work zones, driver population, and calibration. CAF adjustments for turbulence at bottlenecks are best calibrated from local data or, alternatively, are based on regional or state defaults. CAF defaults for weather and incident effects are found in Chapter 11, along with additional discussion on how to apply them. If desired, capacity can be further adjusted to account for unfamiliar drivers in the traffic stream. While the default CAF for driver population is set to 1.0, guidance is provided in Chapter 26 that gives estimates of CAF based on the composition of the driver population.

Chapter 12 provides additional guidance on capacity definitions, while Chapter 26 provides guidance on estimating freeway segment capacity, including weaving segment capacity, from field data.

Ramp–Freeway Junction Capacity Checkpoint

As noted previously, the capacity of the upstream or downstream freeway segment generally limits flow through a merge or diverge area, if the number of freeway lanes entering and leaving the ramp junction is the same. In such cases, the critical checkpoint for freeway capacity is

At a lane drop or lane addition location, capacity of the freeway must be checked both upstream and downstream of the ramp junction.

- Immediately downstream of an on-ramp influence area (v_{FO}), or
- Immediately upstream of an off-ramp influence area (v_F).

These are logical checkpoints, since each represents the point at which maximum freeway flow exists.

When a ramp junction or major merge/diverge area involves lane additions or lane drops at the junction, freeway capacity must be checked both immediately upstream and immediately downstream of the ramp influence area. Failure of any ramp–freeway junction capacity check (i.e., demand exceeds capacity: v/c is greater than 1.00) results in LOS F.

Failure of any ramp–freeway junction capacity check results in LOS F.

In addition, the analyst may perform an off-ramp queue storage ratio check by using the procedures in Chapter 23, Ramp Terminals and Alternative Intersections. If the queue storage ratio exceeds 1.0, the queue may spill back onto the freeway.

Ramp Roadway Capacity Checkpoint

The capacity of the ramp roadway should always be checked against the demand flow rate on the ramp. For on-ramp or merge junctions, this is rarely a problem. Theoretically, cases could exist in which demand exceeds capacity. A failure due to insufficient on-ramp capacity does not, in itself, create problems on the freeway. Rather, it would result in queuing at the streetside terminal of the ramp (or in the case of a freeway-to-freeway ramp, on the entering freeway).

At off-ramps or diverge areas, the most frequent cause of failure is insufficient capacity on the off-ramp—due to either the ramp roadway or a failure of the ramp–street terminal. This methodology checks only for the off-ramp roadway capacity. The capacity of the ramp–street junction must be evaluated by using appropriate methodologies for unsignalized intersections (Chapter 20, 21, or 22) or signalized interchange ramp terminals (Chapter 23).

If the off-ramp demand flow rate v_R exceeds the capacity of the off-ramp, LOS F prevails. If appropriate analysis results in a finding that the ramp–street terminal is operating at a v/c ratio greater than 1.00 on the ramp approach leg, a queuing analysis should be conducted to evaluate (*a*) the extent of the queue that is likely to exist on the ramp roadway and (*b*) whether the queue is close enough to the ramp–freeway junction to affect its operation negatively.

Maximum Desirable Flow Entering the Ramp Influence Area

Failure of the check for flow entering the ramp influence area (v_{12}, v_{R12}) does not automatically result in LOS F but does indicate the need for additional interpretation of the results.

While a checkpoint for v_{12} (off-ramps) or v_{R12} (on-ramps) is conducted, failure does not result in assignment of LOS F unless another failure occurs on a ramp roadway or freeway segment. Failing this checkpoint generally means that there will be more turbulence in the ramp junction influence area than predicted by this methodology. Thus, predicted densities are most likely lower than those that will exist, and predicted speeds are most likely higher than those that will actually occur.

Step 4: Estimate Density in the Ramp Influence Area and Determine the Prevailing LOS

Once the flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area is determined, the expected density in the ramp influence area can be estimated.

Density in On-Ramp (Merge) Influence Areas

The density in on-ramp influence areas is estimated with Equation 14-22:

Equation 14-22

$$D_R = 5.5475 + 0.00734v_R + 0.0078v_{12} - 0.00627L_A$$

where D_R is the density in the ramp influence area (pc/mi/ln) and all other variables are as previously defined.

As more on-ramp vehicles and freeway vehicles in Lanes 1 and 2 enter the ramp influence area, its density is expected to increase. As the length of the acceleration lane increases, there is more space in the ramp influence area, and operating speeds of merging vehicles are expected to increase—both tending to reduce densities.

Density in Off-Ramp (Diverge) Influence Areas

The density in off-ramp influence areas is estimated with Equation 14-23:

Equation 14-23

$$D_R = 4.252 + 0.0086v_{12} - 0.009L_D$$

where all variables are as previously defined.

There is no separate term for v_R because it is included in v_{12} for off-ramps. As the number of vehicles entering the ramp influence area increases, density increases. As the length of the deceleration lane increases, the additional space provided and the resulting higher speeds of merging vehicles both act to reduce density.

Determining LOS

LOS in ramp influence areas is directly related to the estimated density within the area, as given by Equation 14-22 or Equation 14-23. Exhibit 14-3 contains the criteria for this determination. Note again that density definitions of LOS apply only to stable flow (i.e., LOS A–E). LOS F exists only when the capacity of the ramp junction is insufficient to accommodate the existing or projected demand flow rate.

If a merge or diverge segment is determined (or expected) to operate at LOS F, the analyst should go to Chapters 10 and 11, relating to freeway facilities, and conduct a facility analysis that will estimate the spatial and time impacts of queuing resulting from the breakdown.

Step 5: Estimate Speeds in the Vicinity of Ramp–Freeway Junctions

While an estimation of average vehicle speeds within and adjacent to ramp influence areas is not necessary, it is often a useful additional performance measure. Two types of speeds may be estimated:

- Average speed of vehicles within the ramp influence area (mi/h), and
- Average speed of vehicles across all lanes (including outer lanes) within the 1,500-ft length of the ramp influence area (mi/h).

Both types of speeds are needed when a freeway facility analysis is conducted (Chapters 10 and 11). The first type of speed provides a useful companion measure to density within the ramp influence area in all cases.

Exhibit 14-13 and Exhibit 14-14 provide equations for estimating the average speed of vehicles (*a*) within the ramp influence area and (*b*) in outer lanes of the freeway adjacent to the 1,500-ft ramp influence area. For four-lane freeways (two lanes in each direction), there are no "outer lanes." For six-lane freeways (three lanes in each direction), there is one outer lane (Lane 3). For eight-lane freeways (four lanes in each direction), there are two outer lanes (Lanes 3 and 4).

Exhibit 14-13
Estimating Speed at On-Ramp (Merge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS \times SAF - (FFS \times SAF - 42)M_S$ $M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002(L_A \times S_{FR} \times SAF/1{,}000)$	
Outer lanes of freeway	$S_O = FFS \times SAF$	$v_{OA} < 500$ pc/h
	$S_O = FFS \times SAF - 0.0036(v_{OA} - 500)$	$500 \le v_{OA} \le 2{,}300$ pc/h
	$S_O = FFS \times SAF - 6.53 - 0.006(v_{OA} - 2{,}300)$	$v_{OA} > 2{,}300$ pc/h

Exhibit 14-14
Estimating Speed at Off-Ramp (Diverge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS \times SAF - (FFS \times SAF - 42)D_S$ $D_S = 0.883 + 0.00009v_R - 0.013S_{FR} \times SAF$	
Outer lanes of freeway	$S_O = 1.097 \times FFS \times SAF$ $S_O = 1.097 \times FFS \times SAF - 0.0039(v_{OA} - 1{,}000)$	$v_{OA} < 1{,}000$ pc/h $v_{OA} \geq 1{,}000$ pc/h

Note that Exhibit 14-13 and Exhibit 14-14 include the impact of a speed adjustment factor (SAF). The SAF can represent the effects of a combination of different sources, including weather, work zone effects, or driver population. Default SAFs and guidance on applying them are found in Chapter 11.

Exhibit 14-15 provides equations to determine the average speed of all vehicles (ramp plus all freeway vehicles) within the 1,500-ft length of the ramp influence area.

Exhibit 14-15
Estimating Average Speed of All Vehicles at Ramp–Freeway Junctions

Value	Equation
Average flow in outer lanes v_{OA} (pc/h)	$v_{OA} = \dfrac{v_F - v_{12}}{N_O}$
Average speed for on-ramp (merge) junctions (mi/h)	$S = \dfrac{v_{R12} + v_{OA}N_O}{\left(\frac{v_{R12}}{S_R}\right) + \left(\frac{V_{OA}N_O}{S_O}\right)}$
Average speed for off-ramp (diverge) junctions (mi/h)	$S = \dfrac{v_{12} + v_{OA}N_O}{\left(\frac{v_{12}}{S_R}\right) + \left(\frac{V_{OA}N_O}{S_O}\right)}$

While many (but not all) of the variables in Exhibit 14-13 through Exhibit 14-15 have been defined previously, all are redefined here for convenience:

S_R = average speed of vehicles within the ramp influence area (mi/h); for merge areas, this includes all ramp and freeway vehicles in Lanes 1 and 2; for diverge areas, this includes all vehicles in Lanes 1 and 2;

S_O = average speed of vehicles in outer lanes of the freeway adjacent to the 1,500-ft ramp influence area (mi/h);

S = average speed of all vehicles in all lanes within the 1,500-ft length covered by the ramp influence area (mi/h);

FFS = free-flow speed of the freeway (mi/h);

S_{FR} = free-flow speed of the ramp (mi/h);

L_A = length of acceleration lane (ft);

L_D = length of deceleration lane (ft);

v_R = demand flow rate on ramp (pc/h);

v_{12} = demand flow rate in Lanes 1 and 2 of the freeway immediately upstream of the ramp influence area (pc/h);

$v_{R_{12}}$ = total demand flow rate entering the on-ramp influence area, including v_{12} and v_R (pc/h);

v_{OA} = average demand flow per lane in outer lanes adjacent to the ramp influence area (not including flow in Lanes 1 and 2) (pc/h/ln);

4. EXTENSIONS TO THE METHODOLOGY

SPECIAL CASES

As noted previously, the computational procedure for ramp–freeway junctions was calibrated for single-lane, right-side ramps. Many other merge and diverge configurations may be encountered, however. In these cases, the core methodology is modified to account for special situations. These modifications are discussed in the sections that follow.

Single-Lane Ramp Lane Additions and Lane Drops

On-ramps and off-ramps do not always include merge and diverge elements. In some cases, there are lane additions at on-ramps or lane drops at off-ramps. Lane additions and lane drops are defined as merge and diverge segments with acceleration and deceleration lane lengths, respectively, exceeding 1,500 ft.

Analysis of single-lane additions and lane drops is relatively straightforward. The freeway segment downstream of the on-ramp or upstream of the off-ramp is simply considered to be a basic freeway segment with an additional lane. The procedures in Chapter 12, Basic Freeway and Multilane Highway Segments, should be applied in this case.

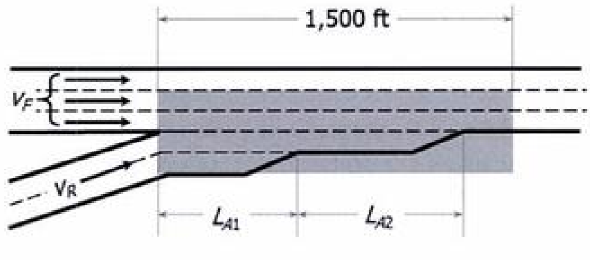
The case of an on-ramp lane addition followed by an off-ramp lane drop may be a weaving segment and should be evaluated with the procedures of Chapter 13, Freeway Weaving Segments. This configuration may be either a weaving segment or a basic segment, depending on the distance between the ramps. Note that some segments may be classified as a weaving segment at higher volumes and as a basic segment at lower volumes.

Ramps with two or more lanes frequently have lane additions or drops for some or all of the ramp lanes. These cases are covered in the following sections.

Two-Lane On-Ramps

Exhibit 14-16 illustrates the geometry of a typical two-lane ramp–freeway junction. It is characterized by two separate acceleration lanes, each successively forcing merging maneuvers to the left.

Exhibit 14-16
Typical Geometry of a Two-Lane Ramp–Freeway Junction



Two-lane on-ramps entail two modifications to the basic methodology: the flow remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area is generally somewhat higher than it is for one-lane on-ramps in similar situations, and densities in the merge influence area are lower than those for similar one-lane on-ramp situations. The lower density is primarily due to the existence of two acceleration lanes and the generally longer distance over which these lanes extend. Thus, two-lane on-ramps handle higher ramp flows more smoothly and at a better LOS than if the same flows were carried on a one-lane ramp–freeway junction.

However, two-lane on-ramp–freeway junctions do not enhance the capacity of the junction. The downstream freeway capacity still controls the total output capacity of the merge area, and the maximum desirable number of vehicles entering the ramp influence area is not changed.

There are three computational modifications to the general methodology for two-lane on-ramps.

First, while v_{12} is still estimated as $v_F \times P_{FM}$, the values of P_{FM} are modified as follows:

- For four-lane freeways, $P_{FM} = 1.000$;
- For six-lane freeways, $P_{FM} = 0.555$; and
- For eight-lane freeways, $P_{FM} = 0.209$.

Second, in all equations using the length of the acceleration lane L_A, that length is replaced by the effective length of both acceleration lanes L_{Aeff} from Equation 14-25:

$$L_{Aeff} = 2L_{A1} + L_{A2}$$

Equation 14-25

A two-lane ramp is always considered to be isolated (i.e., no adjacent ramp conditions affect the computation).

Component lengths are as illustrated in Exhibit 14-16. Some two-lane on-ramps may have acceleration lanes that are longer than the 1,500 ft specified in Exhibit 14-16. In these cases, the acceleration lane length used for calculation should be set to 1,500 ft, since the methodology is not calibrated for greater lengths.

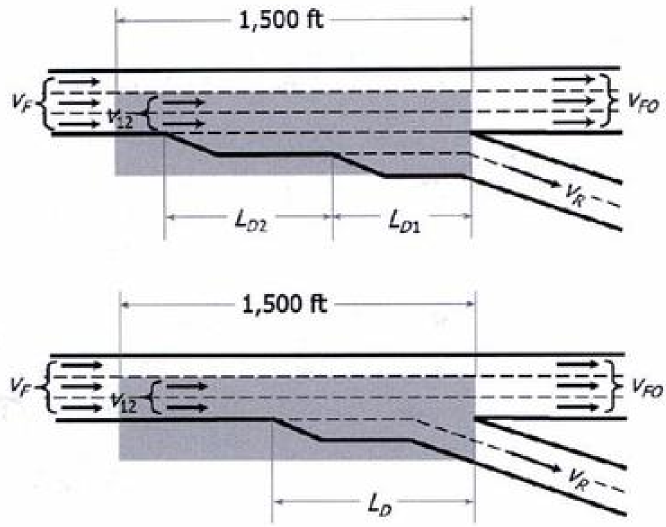
Two-Lane Off-Ramps

Two common types of diverge geometries are in use with two-lane off-ramps, as shown in Exhibit 14-17. In the first, two successive deceleration lanes are introduced. In the second, a single deceleration lane is used. The left-hand ramp lane splits from Lane 1 of the freeway at the gore area, without a deceleration lane.

As is the case for two-lane on-ramps, there are three computational step modifications. While v_{12} is still computed as $v_R + (v_F - v_R) \times P_{FD}$, the values of P_{FD} are modified as follows:

- For four-lane freeways, $P_{FD} = 1.000$;
- For six-lane freeways, $P_{FD} = 0.450$; and
- For eight-lane freeways, $P_{FD} = 0.260$.

Exhibit 14-17
Common Geometries for Two-Lane Off-Ramp–Freeway Junctions



Where a single deceleration lane is used, there is no modification to the length of the deceleration lane L_D; where two deceleration lanes exist, the length is replaced by the effective length L_{Deff} in all equations, obtained from Equation 14-26:

Equation 14-26

$$L_{Deff} = 2L_{D1} + L_{D2}$$

A two-lane ramp is always considered to be isolated (i.e., no adjacent ramp conditions affect the computation).

Component lengths are as illustrated in Exhibit 14-17. Some two-lane off-ramps may have deceleration lanes that are longer than the 1,500 ft specified in Exhibit 14-17. In these cases, the acceleration lane length used for calculation should be set to 1,500 ft, since the methodology is not calibrated for greater lengths.

The capacity of a two-lane off-ramp is essentially equal to that of a similar one-lane off-ramp.

The capacity of a two-lane off-ramp–freeway junction is essentially equal to that of a similar one-lane off-ramp; that is, the total flow capacity through the diverge is unchanged. It is limited by the upstream freeway, the downstream freeway, or the off-ramp capacity. While the capacity is not affected by the presence of two-lane junctions, the lane distribution of vehicles is more flexible than in a similar one-lane case. The two-lane junction may also be able to accommodate a higher off-ramp flow than can a single-lane off-ramp.

Left-Hand On- and Off-Ramps

While they are not normally recommended, left-hand ramp–freeway junctions do exist on some freeways, and they occur frequently on C-D roadways. The left-hand ramp influence area covers the same 1,500-ft length as that of right-hand ramps—upstream of off-ramps and downstream of on-ramps.

For right-hand ramps, the ramp influence area involves Lanes 1 and 2 of the freeway. For left-hand ramps, the ramp influence area involves the two leftmost lanes of the freeway. For four-lane freeways (two lanes in each direction), this does not involve any changes, since only Lanes 1 and 2 exist. For six-lane freeways (three lanes in each direction), the flow in Lanes 2 and 3 (v_{23}) is involved. For eight-lane freeways (four lanes in each direction), the flow in Lanes 3 and 4 (v_{34}) is involved.

While there is no direct methodology for the analysis of left-hand ramps, some rational modifications can be applied to the right-hand ramp methodology to produce reasonable results (*4*).

It is suggested that analysts compute v_{12} as if the ramp were on the right. An estimate of the appropriate flow rate in the two leftmost lanes is then obtained by multiplying the result by the adjustment factors shown in Exhibit 14-18.

Exhibit 14-18
Adjustment Factors for Left-Hand Ramp–Freeway Junctions

Freeway Size	On-Ramps	Off-Ramps
Four-lane	1.00	1.00
Six-lane	1.12	1.05
Eight-lane	1.20	1.10

The remaining computations for density and speed continue by using the value of v_{23} (six-lane freeways) or v_{34} (eight-lane freeways), as appropriate. All capacity values remain unchanged.

Ramp–Freeway Junctions on 10-Lane Freeways (Five Lanes in Each Direction)

Freeway segments with five continuous lanes in a single direction are becoming more common in North America. A procedure is therefore needed to analyze a single-lane, right-hand on- or off-ramp on such a segment.

The approach taken is relatively simple: estimate the flow in Lane 5 of such a segment and deduct it from the approaching freeway flow v_F. With the Lane 5 flow deducted, the segment can now be treated as if it were an eight-lane freeway (*5*). Exhibit 14-19 shows the recommended values for flow rate in Lane 5 of these segments.

Exhibit 14-19
Expected Flow in Lane 5 of a 10-Lane Freeway Immediately Upstream of a Ramp–Freeway Junction

On-Ramps		Off-Ramps	
Approaching Freeway Flow v_F (pc/h)	Approaching Lane 5 Flow v_5 (pc/h)	Approaching Freeway Flow v_F (pc/h)	Approaching Lane 5 Flow v_5 (pc/h)
≥8,500	2,500	≥7,000	0.200 v_F
7,500–8,499	0.285 v_F	5,500–6,999	0.150 v_F
6,500–7,499	0.270 v_F	4,000–5,499	0.100 v_F
5,500–6,499	0.240 v_F	<4,000	0
<5,500	0.220 v_F		

Once the expected flow in Lane 5 is determined, the effective total freeway flow rate in the remaining four lanes is computed from Equation 14-27:

Equation 14-27

$$v_{F4eff} = v_F - v_5$$

where

v_{F4eff} = effective approaching freeway flow in four lanes (pc/h),

v_F = total approaching freeway flow in five lanes (pc/h), and

v_5 = estimated approaching freeway flow in Lane 5 (pc/h).

The remainder of the analysis uses the adjusted approaching freeway flow rate and treats the geometry as if it were a single-lane, right-hand ramp junction on an eight-lane freeway (four lanes in each direction).

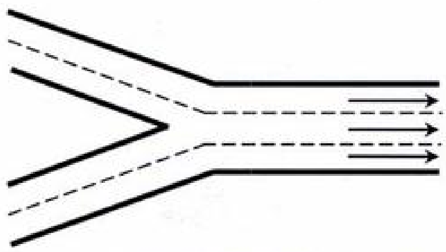
There is no calibrated procedure for adapting the methodology of this chapter to freeways with more than five lanes in one direction. However, the approach of Equation 14-27 is conceptually adaptable to such situations. A local calibration of the amount of traffic using Lanes 5+ would be needed. The remaining flow could then be modeled as if it were taking place on a four-lane (one direction) segment.

Major Merge Areas

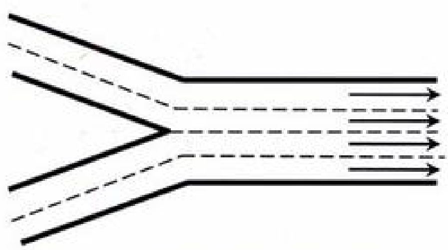
A major merge area is one in which two primary roadways, each having multiple lanes, merge to form a single freeway segment. Such junctions occur when two freeways join to form a single freeway or when a major multilane high-speed ramp joins with a freeway. Major merges are different from one- and two-lane on-ramps in that each of the merging roadways is generally at or near freeway design standards and no clear ramp or acceleration lane is involved in the merge.

Such merge areas come in a variety of geometries, all of which fall into one of two categories. In one geometry, the number of lanes leaving the merge area is one less than the total number of lanes entering it. In the other, the number of lanes leaving the merge area is the same as that entering it. These geometries are illustrated in Exhibit 14-20.

Exhibit 14-20
Major Merge Areas Illustrated



(a) Major Merge with One Lane Dropped



(b) Major Merge with No Lane Dropped

There are no effective models of performance for a major merge area. Therefore, analysis is limited to checking capacities on the approaching legs and the downstream freeway segment. A merge failure would be indicated by a v/c ratio in excess of 1.00.

LOS cannot be determined for major merge areas.

LOS cannot be determined specifically for major merge areas. Problems in major merge areas usually result from insufficient capacity of the downstream freeway basic, merge/diverge, or weaving segment. A rough estimate of LOS in a major merge area could be obtained by applying the basic freeway segment criteria to the segment immediately downstream of the merge. However, this would not account for the effect of turbulence in the segment, and operating conditions would likely be worse than predicted.

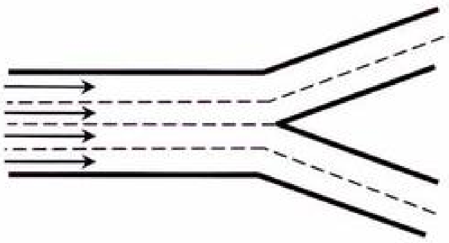
Major Diverge Areas

A major diverge area is one in which two primary roadways, each having multiple lanes, diverge from a single freeway segment. Such junctions occur when a freeway splits to become two separate freeways or when a major multilane high-speed ramp diverges from the freeway. Major diverges are different from one- and two-lane off-ramps in that each of the diverging roadways is generally at or near freeway design standards and no clear ramp or deceleration lane is involved in the merge.

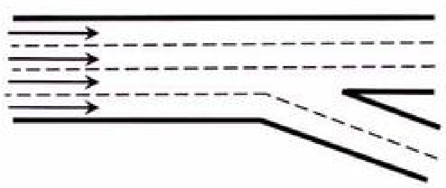
The two common geometries for major diverge areas are illustrated in Exhibit 14-21. In the first case, the number of lanes leaving the diverge area is the same as the number entering it. In the second, the number of lanes leaving the diverge area is one more than the number entering it.

The principal analysis of a major diverge area involves checking the capacity of entering and departing roadways, all of which are generally built to mainline standards. A failure results when any of the demand flow rates exceeds the capacity of the segment.

Exhibit 14-21
Major Diverge Areas Illustrated



(a) Major Diverge Area with No Lane Addition



(b) Major Diverge Area with Lane Addition

For major diverge areas, a model exists for computing the average density across all approaching freeway lanes within 1,500 ft of the diverge, as given in Equation 14-28:

Equation 14-28

$$D_{MD} = 0.0175\left(\frac{v_F}{N}\right)$$

where

D_{MD} = density in the major diverge influence area (which includes all approaching freeway lanes) (pc/mi/ln),

v_F = demand flow rate immediately upstream of the major diverge influence area (pc/h), and

N = number of lanes approaching the major diverge (ln).

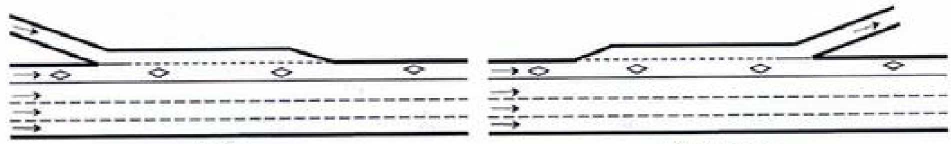
The result can be compared with the criteria of Exhibit 14-3 to determine a LOS for the major diverge influence area. Note that the density and LOS estimates are only valid for stable cases (i.e., not in cases in which LOS F exists because of a capacity deficiency on the approaching or departing legs of the diverge).

MANAGED LANE ACCESS POINTS

Managed lanes on freeways may be accessed in many ways. One possible design is the provision of direct entries and exits to a managed lane or lanes by ramps. This is illustrated in Exhibit 14-22.

These merge or diverge segments onto a one-lane managed lane facility may be treated as isolated merge and diverge areas onto a one-lane mainline and evaluated by using an adaptation of the methods in this chapter. This accounts for the fact that there is no interaction between general purpose lanes and the managed lane in the vicinity of the ramp. Since the procedures of this chapter have been calibrated to segments with two or more lanes on a mainline segment, a modification to the inputs is needed.

Exhibit 14-22
Direct Ramp Access to Managed Lanes



(a) Merge (b) Diverge

Managed lane segment types were defined in Chapter 10.

The operations of a managed lane (ML) merge or ML diverge segment with a single mainline lane can be approximated by doubling the managed lane mainline volume before analysis and evaluating the segment as if there were two through lanes on the managed lanes. The resulting computational results for segment speed and density will then be true to the assumptions used in development of the methods in this chapter. The results should then be applied only to the single managed lane.

Care should be taken to consider only the single managed lane in performing a capacity check on the segment. For the on-ramp case, the capacity of the ramp roadway and the downstream managed lane should be compared with demand flows. For the off-ramp case, the capacities of the ramp roadway and the upstream managed lane are used. Where either capacity is exceeded by demand, a failure (LOS F) is anticipated. The capacity of the ML merge or ML diverge segment should further be capped to not exceed the capacity of a basic managed lane segment, especially where there is an adjacent friction effect on managed lane operations.

For managed lane segments with more than one through lane, the procedures in this chapter can be applied without further adjustments to estimate the capacity, segment speed, and other performance measures for the ML merge or ML diverge segment. However, care should be taken when an overall managed lane facility is being evaluated and the separation between the managed lane and general purpose lanes requires consideration of the adjacent friction effect, as described in Chapter 12, Basic Freeway and Multilane Highway Segments. In these cases, the core freeway facilities methodology in Chapter 10 offers additional adjustments.

EFFECT OF RAMP CONTROL AT RAMPS

For the purposes of this methodology, procedures are not modified in any way to account for the local effect of ramp control—except for the limitation that the ramp meter may have on the ramp demand flow rate. Research (7) has found that the breakdown of a merge area may be a probabilistic event based on the platoon characteristics of the arriving ramp vehicles. Ramp meters facilitate uniform gaps between entering ramp vehicles and may reduce the probability of a breakdown on the associated freeway mainline.

5. APPLICATIONS

The methodology of this chapter is most often used to estimate the capacity and LOS of ramp–freeway junctions. The steps are most easily applied in the operational analysis mode (i.e., all traffic and roadway conditions are specified), and the capacity (and v/c ratio) and expected LOS are found. Other types of analysis are also possible.

EXAMPLE PROBLEMS

The following example problems illustrating the application of the methodology of this chapter are found in Chapter 28, Freeway Merge and Diverge Segments: Supplemental:

- Isolated, single-lane, right-hand on-ramp to a four-lane freeway;
- Two adjacent single-lane, right-hand off-ramps on a six-lane freeway;
- Single-lane on-ramp followed by a one-lane off-ramp on an eight-lane freeway;
- Single-lane left-hand on-ramp on a six-lane freeway; and
- Service flow rates and service volumes for an isolated on-ramp on a six-lane freeway.

RELATED CONTENT IN THE HCMAG

The *Highway Capacity Manual Applications Guide* (HCMAG), accessible through the online HCM Volume 4, provides guidance on applying the HCM on freeway merge and diverge segments. Case Study 4 goes through the process of identifying the goals, objectives, and analysis tools for investigating LOS on New York State Route 7, a 3-mi route north of Albany. The case study applies the analysis tools to assess the performance of the route, to identify areas that are deficient, and to investigate alternatives for correcting the deficiencies.

This case study includes the following problems related to freeway merge and diverge segments:

1. Problem 2: Analysis of a complex interchange on the western end of the route.
    a. Subproblem 2c: Ramp and ramp junction LOS for the on-ramp from Alternate Route 7 to I-87 northbound
    b. Subproblem 2d: Mitigation techniques for the on-ramp from Alternate Route 7 to I-87 northbound
2. Problem 3: Weaving and ramp analysis
    a. Subproblem 3b: Freeway ramp analysis
    b. Subproblem 3c: Nonstandard ramp and weave analysis in the southwestern quadrant

Other problems in the case study evaluate the operations of freeway merge and diverge segments as part of a greater freeway facility as discussed in the methodology in Chapter 10, Freeway Facilities Core Methodology.

Although the HCMAG was based on the HCM2000's procedures and chapter organization, the general thought process described in its case studies is also applicable to this edition of the HCM.

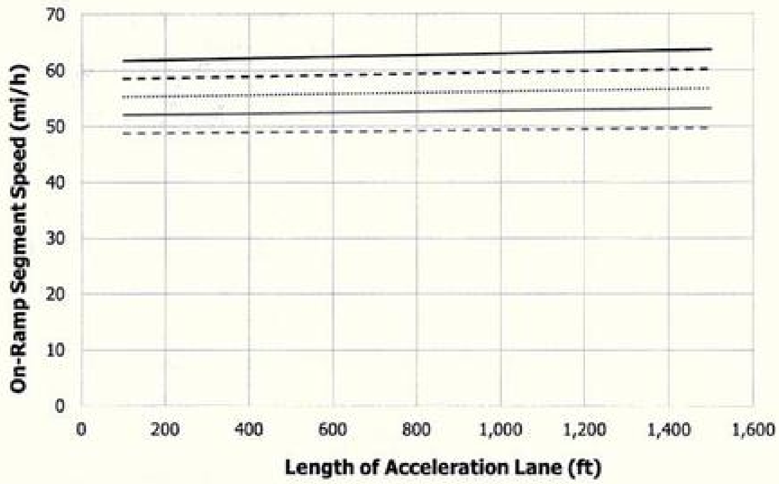
EXAMPLE RESULTS

This section presents the results of applying this chapter's method in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs, as well as to help evaluate whether their analysis results are reasonable. The exhibits in this section are not intended to substitute for an actual analysis and are deliberately provided in a format large enough to depict general trends in the results, but not large enough to pull out specific results.

Sensitivity of Merge Results to Length of Acceleration Lane

Exhibit 14-23 presents illustrative results of the effect of acceleration lane length on the overall speed in a merge segment. Results are shown for five values of FFS from 55 mi/h to 75 mi/h.

Exhibit 14-23
Illustrative Effect of Acceleration Lane Length on Merge Segment Speed



Note: Calculated by using this chapter's method, assuming 3 lanes per direction, on-ramp FFS = 40 mi/h, mainline demand = 5,000 veh/h, and ramp volume = 1,000 veh/h.

The results illustrate that an increase in the acceleration lane length increases the overall speed of the merge segment slightly. This is explained practically, because greater acceleration lane length gives vehicles more space for completing the merge maneuver. In the methodology, the added acceleration lane length further translates to a reduced density, since the acceleration lane is included in the total lane miles in the segment. Higher free-flow speeds result in higher segment speeds uniformly across the range of acceleration lane length.

Sensitivity of Merge Results to Traffic Demand Level

Exhibit 14-24 presents illustrative results of the effect of increasing traffic demand on the overall speed in a merge segment. The on-ramp demand was assumed at a fixed ratio of 10% of mainline flow. Results are shown for five values of FFS from 55 mi/h to 75 mi/h.

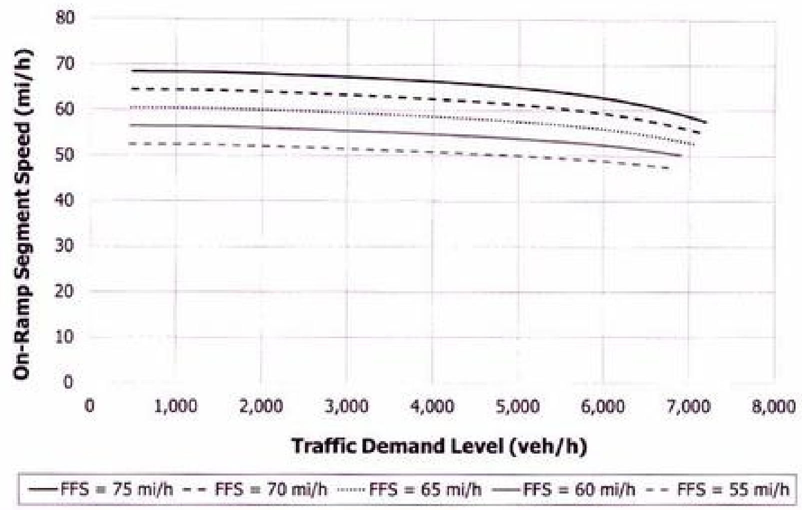


Exhibit 14-24
Illustrative Effect of Traffic Demand Level on Merge Segment Speed

Note: Calculated by using this chapter's method, assuming 3 lanes per direction, on-ramp FFS = 45 mi/h, acceleration lane length = 300 ft, and on-ramp demand set to 10% of mainline demand.

The results illustrate that an increase in traffic demand level decreases the overall speed of the merge segment. Higher traffic demand (at a fixed segment capacity) results in a greater density of vehicles and decreased headways between vehicles. At greater densities, drivers respond by reducing their travel speed. Higher FFS values shift the entire speed–flow relationship upward.

Sensitivity of Diverge Results to Deceleration Lane Length

Exhibit 14-25 presents illustrative results of the effect of the length of the deceleration lane on the overall speed in a diverge segment. Results are shown for five values of FFS from 55 mi/h to 75 mi/h.

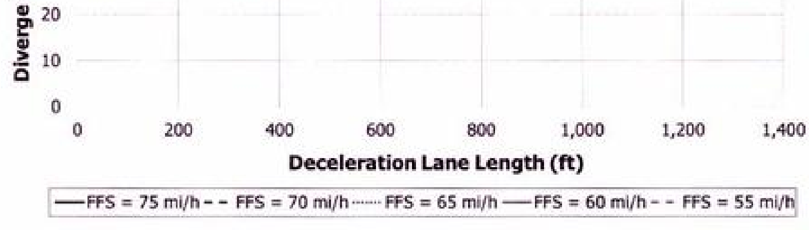


Exhibit 14-25
Illustrative Effect of Deceleration Lane Length on Diverge Segment Speed

Note: Calculated by using this chapter's method, assuming 4 lanes per direction, off-ramp FFS = 40 mi/h, mainline demand = 8,000 veh/h, and off-ramp demand = 800 veh/h.

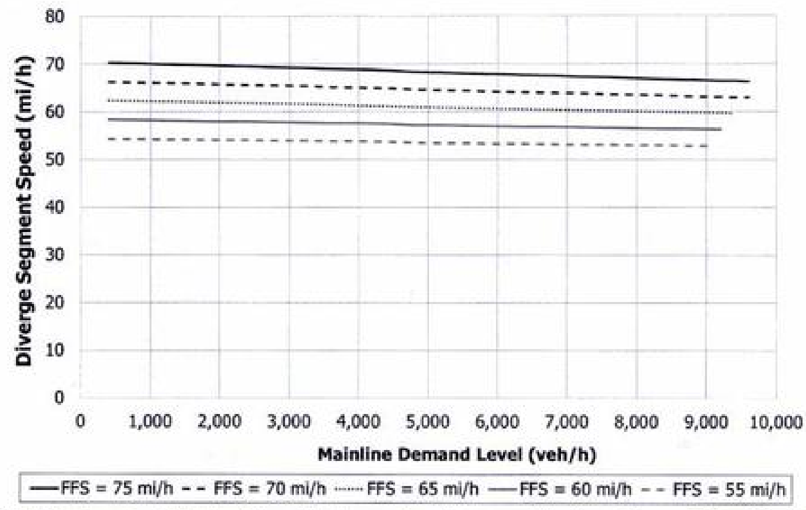
The results illustrate that an increase in the deceleration lane length has virtually no effect on the estimated segment speed. In the methodology, the deceleration lane length is used only to estimate segment density but does not appear in the equation for average segment speed. Higher free-flow speeds result in higher segment speeds uniformly across the range of acceleration lane length.

Sensitivity of Diverge Results to Traffic Demand Level

Exhibit 14-26 presents illustrative results of the effect of increasing traffic demand on the overall speed in a diverge segment. The off-ramp demand was assumed at a fixed ratio of 20% of mainline flow. Results are shown for five values of free-flow speed from 55 mi/h to 75 mi/h.

The results illustrate that an increase in traffic demand level decreases the overall speed of the diverge segment. Higher traffic demand (at a fixed segment capacity) results in a greater density of vehicles and decreased headways between vehicles. At greater densities, drivers respond by reducing their travel speed. Higher FFS values shift the entire speed–flow relationship upward.

Exhibit 14-26
Illustrative Effect of Traffic Demand Level on Diverge Segment Speed



Note: Calculated by using this chapter's method, assuming 4 lanes per direction, off-ramp FFS = 40 mi/h, deceleration lane length = 500 ft, and off-ramp demand = 20% of mainline demand.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design analysis.

Establish Analysis Boundaries

No ramp–freeway junction is completely isolated. However, for the purposes of this methodology, many may operate as if they were. In the analysis of ramp–freeway junctions, establishing the segment of freeway over which ramp junctions are to be analyzed is important. Once this is done, each ramp may be analyzed in conjunction with the possible impacts of upstream and downstream adjacent ramps according to the methodology.

Analysis boundaries may also include different demand scenarios related to the time of the day or to different development scenarios that produce different demand flow rates.

Any application of the methodology presented in this chapter can be made easier by carefully defining the spatial and time boundaries of the analysis.

Operational Analysis

Operational analysis determines density, LOS, and speed within the ramp influence area for a specified set of conditions.

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including

- Analysis hour demand volumes for the subject ramp, adjacent ramps, and freeway (veh/h);
- Heavy vehicle percentages for all component demand volumes (ramps, adjacent ramps, freeway);
- PHF for all component demand volumes (ramp, adjacent ramps, freeway);
- Freeway terrain (level, rolling, mountainous, specific grade);
- FFS of the freeway and ramp (mi/h);
- Ramp geometrics: number of lanes, terrain, length of acceleration lane(s) or deceleration lane(s); and
- Distance to upstream and downstream adjacent ramps (ft).

The outputs of an operational analysis will be estimates of density, LOS, and speed for the ramp influence area. The capacity of the ramp–freeway junction will also be established.

The steps of the methodology, described in the Methodology section, are to be followed directly without modification.

Design Analysis

Design analysis seeks to determine the geometric characteristics of the ramp that are needed to deliver a target LOS.

In design analysis, a target LOS is set and all relevant demand volumes are specified. The analysis seeks to determine the geometric characteristics of the ramp that are needed to deliver the target LOS. These characteristics include

- FFS of the ramp S_{FR} (mi/h),
- Length of acceleration L_A or deceleration lane L_D (ft), and
- Number of lanes on the ramp.

In some cases, variables such as the type of junction (e.g., major merge, two-lane) may also be under consideration.

There is no convenient way to compute directly the optimal value of any one variable without specifying all of the others. Even then, the computational methodology does not easily create the desired result.

Therefore, most design analysis becomes a trial-and-error application of the operational analysis procedure. Individual characteristics can be incrementally changed, as can groups of characteristics, to find scenarios that produce the desired LOS.

In many cases, some of the variables may be fixed by site-specific conditions. These can be set at their limiting values before an attempt is made to optimize the others.

A spreadsheet can be programmed to complete such an analysis. Scenario results are provided by simply changing some of the input variables under consideration. HCM-implementing software can also be used to simplify the computational process.

Planning and Preliminary Engineering Analysis

Planning and preliminary engineering analysis also seeks to determine the geometric characteristics of the ramp that are needed to deliver a target LOS, but it relies on more general input data.

The desired outputs of planning and preliminary engineering analysis are virtually the same as those for design analysis. The primary difference is that planning and preliminary engineering analysis occurs very early in the process of project consideration.

The first criterion that categorizes such applications is the need to use more general estimates of input data. Many of the default values specified in Chapter 12, Basic Freeway and Multilane Highway Segments; Chapter 13, Freeway Weaving Segments; and this chapter would be applied; alternatively, local default values can be substituted. Demand volumes might be specified only as expected values of annual average daily traffic (AADT) for a target year. Directional design-hour volumes are based on AADTs; default (local or global) values are used for the *K*-factor (the proportion of AADT occurring in the peak hour) and the *D*-factor (the proportion of peak hour traffic traveling in the peak direction). Guidance on these values is given in Chapter 3, Modal Characteristics.

On the basis of these default and estimated values, the analysis is conducted in the same manner as a design analysis.

Service Volumes and Service Flow Rates

The method can be applied to determine service volumes for LOS A–E for a specified set of conditions.

Service volume is the maximum hourly volume that can be accommodated without exceeding the limits of the various levels of service during the worst 15 min of the analysis hour. Service volumes can be found for LOS A–E. LOS F, which represents unstable flow, does not have a service volume.

Service flow rates are the maximum rates of flow (within a 15-min period) that can be accommodated without exceeding the limits of the various levels of service. As is the case for service volumes, service flow rates can be found for LOS A–E, but none is defined for LOS F. The relationship between a service volume and a service flow rate is as follows:

Equation 14-29

$$SV_i = SF_i \times PHF$$

where

SV_i = service volume for LOS i (pc/h),

SF_i = service flow rate for LOS i (pc/h), and

PHF = peak hour factor.

For ramp–freeway junctions, service flow rate or service volume could be defined in several ways. It might be argued that since ramp–freeway junction capacities are usually limited by the upstream or downstream freeway segment, service flow rates and service volumes should be based on basic freeway criteria applied to the upstream or downstream freeway segments. This, however, would ignore the levels of service defined for the ramp influence area, which are the only unique service descriptors for ramps.

Levels of service for ramp–freeway junctions are defined in Exhibit 14-3 and relate to the density within the ramp influence area. The methodology estimates this density by using a series of algorithms affected by demand flows on the freeway, ramp, and adjacent ramps; ramp geometrics; and distances to adjacent ramps. The methodology uses demand volumes in vehicles per hour converted to demand flow rates in passenger cars per hour. Therefore, service flow rates and service volumes would originally be estimated in terms of flow rates in passenger cars per hour. They would then be converted back to demand volumes in vehicles per hour.

Because the balance of ramp and freeway demands has a significant impact on densities, there are several ways to consider service flow rates and volumes:

- The limiting total upstream demand volume that produces a given LOS within the ramp influence area. The split between arriving freeway volume and ramp volume would have to be specified.
- The limiting volume entering the ramp influence area that produces a given LOS within the ramp influence area. Since this relies on the approaching freeway volume, the split between freeway and ramp demand would still have to be specified.
- The limiting ramp volume that produces a given LOS within the ramp influence area, based on a fixed upstream freeway demand.

All of these concepts are viable for establishing a ramp service flow rate or service volume.

In addition to different ways of interpreting a service volume or service flow rate, a large number of characteristics will influence the result, including the PHF, percentage of heavy vehicles, length of acceleration or deceleration lane(s), ramp FFS, and any relevant data for adjacent ramps. Therefore, defining a representative "typical" case with broadly applicable results is virtually impossible. Each case must be individually considered. Chapter 28, Freeway Merges and Diverges: Supplemental, includes an example of how ramp junction service flow rates and volumes can be computed.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for applying alternative tools to the analysis of ramps and ramp junctions. Additional information on this topic may be found in the Volume 4 Technical Reference Library.

The HCM methodology for analyzing merge and diverge segments estimates the density of the ramp influence area (which includes the two rightmost lanes of the freeway and the acceleration or deceleration lane) and provides the respective LOS. As an intermediate step, the methodology estimates the capacity at various points through the section, and if the capacity is exceeded, the LOS is determined to be F without further calculation of density. The methodology is primarily based on the estimation of the demand into the influence area v_{12}.

Since the HCM methodology for analysis of merge and diverge segments has been calibrated on the basis of extensive field data, the method serves as a good comparison and calibration aid for alternative tools, to ensure that merge and diverge segment operations are modeled consistently with this chapter's expectations.

Limitations of the HCM Procedures That Might Be Addressed by Alternative Tools

A listing of the HCM's limitations for freeway merge and diverge is provided in Exhibit 14-27.

Exhibit 14-27
Limitations of the HCM Ramps and Ramp Junctions Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Managed lanes, such as HOV lanes, as ramp entrance lanes	Modeled explicitly by simulation
Ramp metering	Modeled explicitly by simulation
Oversaturated conditions (*Refer to Chapters 10 and 11 for further discussion*)	Modeled explicitly by simulation
Posted speed limit and extent of police enforcement	Can be approximated by using assumptions related to the desired speed along a given segment
Presence of intelligent transportation system features	Several features modeled explicitly by simulation; others may be approximated by using assumptions (for example, by modifying origin–destination demands by time interval)
Freeway operational analysis beyond the 1,500-ft area of influence	Modeled explicitly by simulation
Capacity-enhancing effects of ramp metering	Can be approximated by using assumptions related to car-following, lane-changing, and gap-acceptance behavior

Ramp junctions can also be analyzed with a variety of stochastic and deterministic simulation packages that address freeways. These packages can be useful in analyzing the extent of congestion when there are failures either within or downstream of the simulated facility range.

Additional Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the capacity, speed, and density in the area of influence of on- and off-ramps, given traffic demands and segment characteristics. Alternative tools offer additional performance measures including delay, stops, queue lengths, fuel consumption, pollution, and operating costs. In addition, alternative tools can readily be used to estimate travel time for ramp junctions, which is not a performance measure available through this chapter (but which can be obtained from Chapter 10).

As with most other HCM procedural chapters, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of downstream conditions on lane utilization and backup beyond the segment boundary is a good example of a situation that can benefit from the increased insight offered by a microscopic model.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The subject of performance measure comparisons was discussed in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results. This section deals with topics that apply specifically to ramps and ramp junctions.

When alternative tools are used, the analyst must be careful to note the definitions of simulation outputs. This chapter's measure of effectiveness for ramps and ramp junctions is the density of the ramp influence area. However, most simulators do not provide density estimates separately for the two rightmost lanes within a link. This is a potentially significant obstacle in obtaining the service measures for ramp junctions from a simulator (unless the freeway has only two lanes per direction). Furthermore, in a simulator, there are lane changes along the entire segment. Therefore, how a simulator should address the partial presence of vehicles in the link to ensure compatibility with the HCM is not clear. Also, as is generally the case for basic freeway segments, increased speed variability in driver behavior (which simulators usually include) results in lower average space mean speed and higher density.

Most simulation packages do not provide separate density estimates for the two right-hand lanes within a link, which is a potentially significant obstacle in obtaining service measures.

In obtaining density from alternative models, the following should be considered:

- The ability of the simulator to provide density for the two rightmost lanes of the freeway;
- The vehicles included in the density estimation and how partial presence of vehicles on the link is considered;
- The manner in which the acceleration and deceleration lanes are considered in the density estimation;
- The units used by the simulator to measure density [most use vehicles rather than passenger cars; converting vehicles to passenger cars by using the HCM's passenger car equivalence (PCE) values is typically not appropriate, given that simulator assumptions with regard to heavy vehicle performance vary widely];
- The units used in the reporting of density (i.e., whether density is reported per lane mile);
- The homogeneity of the analysis segment in the simulator, since the HCM assumes conditions to be homogeneous (unless it is a specific upgrade or downgrade segment, in which case the segment length is used to estimate the PCE values); and
- The treatment of driver variability by the simulator, since increased driver variability in the simulator will generally increase the average density.

The HCM provides capacity estimates in units of passenger cars per hour per lane as a function of FFS for the locations approaching and departing the merge junction. In comparing the HCM estimates with capacity estimates from a simulator, the following should be considered:

- The manner in which a simulator provides the number of vehicles exiting a segment may require the provision of virtual detectors at specific points on the simulated segment in some cases so that the maximum throughput can be obtained.
- The simulator provides the maximum throughput at a particular location in units of vehicles rather than passenger cars. Converting these units to passenger cars by using the HCM's PCE values is typically not appropriate, given that simulator assumptions with regard to heavy vehicle performance vary widely.
- A simulator will likely include inputs such as the "minimum separation of vehicles," which greatly affects the maximum throughput.

Conceptual Differences Between the HCM and Simulation Modeling That Preclude Direct Comparison of Results

Ramp junction density does not change with FFS in the HCM method, but density is a function of FFS in most simulation packages.

In the HCM, the density at a ramp junction does not change with FFS, although density drops as a function of FFS on basic freeway segments. In simulators, the density typically changes as a function of FFS (or the desired speed). Therefore, calibration of a site using a specific FFS does not necessarily ensure that the site will be calibrated for a different FFS. Capacity, on the other hand, increases in the HCM with increasing FFS, which is typically the case with simulators.

The HCM method is based on the estimated demand approaching the ramp influence area. This demand is estimated as a function of the presence of and demands on the upstream and downstream ramps. Traffic simulators do not typically allow the user to input the specific percentages of traffic on each lane at the beginning of a link. Their internal rules relative to the lane chosen by a vehicle in a given link vary widely and can be modified by changing various default values within the simulator. In some simulators, virtual vehicles are "aware" of their ultimate destination; in others, the exit choice is made on a link-by-link basis. Therefore, in comparing HCM results with those of a simulator, the analyst should, as an intermediate check, compare the flow approaching the two rightmost lanes of the junction.

Adjustment of Simulation Parameters to the HCM Results

The most important elements to be adjusted in analyzing a ramp junction are as follows:

- The flow approaching the two rightmost lanes (this is an intermediate step but would ensure that the influence of upstream and downstream ramps is considered in a manner compatible with the HCM), and
- The capacity of the junction at the critical locations indicated in the HCM (i.e., downstream of the junction and approaching the influence area).

Step-by-Step Recommendations for Applying Alternative Tools

The following steps are recommended when an alternative tool is applied to the analysis of ramps and ramp junctions:

1. Determine whether the chosen tool can provide density for the two rightmost lanes of the freeway and what approach is used to obtain it (including the treatment of the partial presence of vehicles on the link).
2. Determine the FFS of the study site either from field data or by estimating it according to the Chapter 12 method for basic freeway segments.
3. Enter all available input characteristics (both geometric and traffic characteristics) into the simulator. The length of the segment or link to be simulated should be 1,500 ft, to correspond to the HCM-defined area of influence. Install virtual detectors within the area of influence and at the downstream end of the study segment to obtain density, speeds, and flows.
4. Load the study network above capacity to obtain the maximum throughput, and compare the result with the HCM estimate. Calibrate the simulator by modifying parameters related to the minimum time headway so that the simulated capacity matches the HCM estimate. Estimate the number of simulation runs that will need to be conducted to produce a statistically valid comparison.
5. Compare the flow approaching the two rightmost lanes with the HCM's estimate. Adjust the simulation parameters related to driver awareness of upcoming turns to match the HCM-predicted v_{12} value.

Example Problems Illustrating Alternative Tool Applications

Chapter 28, Freeway Merges and Diverges: Supplemental, includes two example problems that examine situations beyond the scope of this chapter's methodology by using a typical microsimulation-based tool. Both problems are based on that chapter's Example Problem 3, which analyzes an eight-lane freeway segment with an entrance and an exit ramp. The first problem evaluates the effects of the addition of ramp metering, and the second evaluates the impacts of converting the leftmost lane of the mainline into an HOV lane.

6. REFERENCES

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

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CHAPTER 15
TWO-LANE HIGHWAYS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

OVERVIEW

Two-lane highways have one lane for the use of traffic in each direction. The principal characteristic that distinguishes two-lane highway operation from that of other uninterrupted-flow facilities is that passing maneuvers take place in the opposing lane of traffic. Passing maneuvers are limited by the availability of gaps in the opposing traffic stream and by the availability of sufficient sight distance for a driver to discern the approach of an opposing vehicle safely. As demand flows and geometric restrictions increase, opportunities to pass decrease. This creates platoons within the traffic stream, with trailing vehicles subject to additional delay because of the inability to pass the lead vehicles.

Two-lane highways have one lane for the use of traffic in each direction. Passing takes place in the opposing lane of traffic when sight distance is appropriate and safe gaps in the opposing traffic stream are available.

The relationship between passing demand and passing capacity on a two-lane highway is complex. In any given direction, passing demand increases as flows increase. Passing capacity decreases as opposing flows increase. This creates a unique situation on two-lane highways. Operational quality deteriorates at relatively low flows in comparison with multilane facilities. This is because of the compounded impact of generally increasing flows on passing, which affects percent time-spent-following (PTSF), a principal measure of service quality on such highways.

CHAPTER ORGANIZATION

Chapter 15 presents methodologies for analyzing two-lane highway operations under uninterrupted-flow conditions. Uninterrupted flow exists when there are no traffic control devices that interrupt traffic and where no platoons are formed by upstream traffic signals. In general, any segment that is 2.0 to 3.0 mi from the nearest signalized intersection fits into this category. When traffic signals are less than 2.0 mi apart, the facility should be classified as an urban street and analyzed with the methodologies of Chapter 16, Urban Street Facilities, and Chapter 18, Urban Street Segments, which are located in Volume 3.

Section 2 of this chapter presents the following aspects of two-lane highways: typical functions in the highway system, the three classes of highways used in *Highway Capacity Manual* (HCM) methods, speed–flow relationships, and quality of service concepts for motorized vehicles and bicycles.

Section 3 presents a method for evaluating motorized vehicle operations on two-lane highways without passing lanes. This method generates the following performance measures:

- Average travel speed (ATS);
- Ratio of ATS to free-flow speed (FFS);
- PTSF;
- Level of service (LOS) based on one or more of the above measures, depending on the highway class;
- Average travel time;
- Volume-to-capacity (v/c) ratio;

- Total vehicle miles traveled (VMT) during the analysis period; and
- Total vehicle hours of travel (VHT) during the analysis period.

Section 4 extends the core motorized vehicle method presented in Section 3 to incorporate the effects of passing lanes on two-lane highway operations. This section also discusses the effects of various design treatments on two-lane highway operations.

Section 5 presents a method for evaluating bicycle operations on two-lane and multilane highways. The method is applicable to bicycle operations in a shared lane, bicycle lane, or shoulder bikeway. This method generates two performance measures: (*a*) a bicycle LOS score reflecting bicyclist perceptions of operating conditions and (*b*) a bicycle LOS letter based on the bicycle LOS score. Both the bicycle LOS score and letter are comparable with similar scores and letters produced for urban streets in HCM Chapters 16 and 17. Bicycle operations on exclusive- or shared-use paths separate from the highway can be evaluated by using the methods in Chapter 24, Off-Street Pedestrian and Bicycle Facilities.

Section 6 presents guidance on using the results of a two-lane highway analysis, including example results from the methods, information on the sensitivity of results to various inputs, and a service volume table for two-lane highways.

RELATED HCM CONTENT

Other HCM content related to this chapter includes the following:

- Chapter 3, Modal Characteristics, where the Variations in Demand subsection of the Motorized Vehicle Mode section describes typical travel demand patterns for two-lane highways;
- Chapter 4, Traffic Operations and Capacity Concepts, which provides background for the refinements specific to two-lane highways presented in this chapter's Section 2;
- Chapter 26, Freeway and Highway Segments: Supplemental, where Section 2 presents state-specific heavy-vehicle percentages, Section 4 provides example problems with step-by-step calculations using this chapter's methods, and Appendix B presents a method for evaluating the capacity of work zones on two-lane highways;
- Case Study 3, Krome Avenue, in the *HCM Applications Guide* in Volume 4, which demonstrates how this chapter's methods can be applied to the evaluation of an actual two-lane highway; and
- Section J, Two-Lane Highways, in the *Planning and Preliminary Engineering Applications Guide to the HCM,* found in Volume 4, which describes how to incorporate this chapter's methods and performance measures into a planning effort.

2. CONCEPTS

CHARACTERISTICS OF TWO-LANE HIGHWAYS

Functions of Two-Lane Highways in Highway Systems

Two-lane highways are a key element in the highway systems of most states and counties. They are located in many geographical areas and serve a variety of traffic functions. Two-lane highways also serve a number of bicycle trips, particularly recreational trips. Any consideration of operating quality criteria must account for these disparate functions.

Operational criteria for two-lane highways must consider the varying functions they provide as well as the corresponding driver expectations.

Efficient mobility is the principal function of major two-lane highways that connect major trip generators or serve as primary links in state and national highway networks. These routes tend to serve long-distance commercial and recreational travelers, and long sections may pass through rural areas without traffic control interruptions. Consistent high-speed operations and infrequent passing delays are desirable for these types of facilities.

Other paved, two-lane rural highways primarily provide *accessibility* to remote or sparsely populated areas. Such highways provide reliable all-weather access and often serve low traffic demands. Cost-effective access is a primary concern. Although high speed is beneficial, it is not the principal objective. Delay, as indicated by platoon formation, is a more relevant measure of service quality.

Two-lane roads also serve *scenic and recreational* areas in which the vista and environment are meant to be experienced and enjoyed without traffic interruption or delay. High-speed operation is neither expected nor desired. However, passing delays significantly distract from the scenic enjoyment of trips and should be minimized whenever possible.

Two-lane roads may also pass through and serve *small towns and communities*. Such areas have higher-density development than would normally be expected along a rural highway, and speed limits in these areas are often lower. In these cases, drivers expect to be able to maintain speeds close to the posted limit. Since two-lane highway segments serving such developed areas are usually of limited length, passing delays are not a significant issue.

Two-lane highways serve a wide range of functions and a variety of rural areas, as well as more developed areas. Therefore, this chapter's methodology and LOS criteria provide flexibility to encompass the resulting range of driver expectations.

Classification of Two-Lane Highways

Because of the wide range of functions served by two-lane highways, the core motorized vehicle methodology in Section 3 establishes three classes of highways.

The first two classes address *rural two-lane highways*. The methodology for them was developed as part of National Cooperative Highway Research Program (NCHRP) Project 3-55(3) in 1999 (*1*) and revised as part of NCHRP Project 20-7(160) in 2003 (*2*).

There are three classes of two-lane highways:

- *Class I, where motorists expect to travel at relatively high speeds;*
- *Class II, where motorists do not necessarily expect to travel at high speeds; and*
- *Class III, which are highways passing through moderately developed areas.*

The third class addresses two-lane highways in *developed areas*. The analysis approach for these highways is a modification of the rural highway method and was developed by the Florida Department of Transportation (3). This modification has not been subjected to a national calibration study. It is presented here as an alternative procedure, since it is based entirely on Florida data. For clarity, the material is integrated into the overall method and is not discussed separately as an alternative procedure.

The three classes of two-lane highways are defined as follows and illustrated in Exhibit 15-1:

- *Class I two-lane highways* are highways where motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks are generally assigned to Class I. These facilities serve mostly long-distance trips or provide the connections between facilities that serve long-distance trips.
- *Class II two-lane highways* are highways where motorists do not necessarily expect to travel at high speeds. Two-lane highways that are access routes to Class I facilities, that serve as scenic or recreational routes (and not as primary arterials), or that pass through rugged terrain (where high-speed operation would be impossible) are assigned to Class II. These facilities most often serve relatively short trips, the beginning or ending portions of longer trips, or trips for which sightseeing plays a significant role.
- *Class III two-lane highways* serve moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns or developed recreational areas. Local traffic often mixes with through traffic on these segments, and the number of unsignalized driveways and cross-streets is noticeably higher than in a purely rural area. Class III highways can include longer roadway segments passing through more spread-out recreational areas, also with increased roadside densities. Such segments are often accompanied by reduced speed limits that reflect the higher activity level.

The two-lane highway classes are defined on the basis of their function. Most rural two-lane arterials and trunk roads would be considered to be Class I highways, while most two-lane collectors and local roads would be considered to be Class II or Class III highways. However, the primary determinant of a facility's classification is the motorist's expectation, which might not agree with the route's overall functional category. For example, a major intercity route passing through a rugged mountainous area might be described as Class II if drivers recognize that high-speed operation is not feasible due to the terrain, but the route could still be considered to be in Class I.

Exhibit 15-1
Two-Lane Highway Classifications Illustrated

(a) Examples of Class I Two-Lane Highways

(b) Examples of Class II Two-Lane Highways

(c) Examples of Class III Two-Lane Highways

Base Conditions

The base conditions for two-lane highways are the absence of restrictive geometric, traffic, or environmental factors. Base conditions are not the same as typical or default conditions, both of which may reflect common restrictions. Base conditions are closer to what may be considered as ideal conditions (i.e., the best conditions that can be expected given normal design and operational practice). This chapter's methodology accounts for the effects of geometric, traffic, and environmental factors that are more restrictive than the base conditions. The base conditions for two-lane highways are as follows:

- Lane widths greater than or equal to 12 ft,
- Clear shoulders wider than or equal to 6 ft,
- No no-passing zones,
- All passenger cars (i.e., no trucks) in the traffic stream,

- Level terrain, and
- No impediments to through traffic (e.g., traffic signals, turning vehicles).

Traffic can operate ideally only if lanes and shoulders are wide enough not to constrain speeds. Lanes narrower than 12 ft and shoulders narrower than 6 ft have been shown to reduce speeds.

The length and frequency of no-passing zones are a result of the roadway's horizontal and vertical alignment. No-passing zones may be marked by barrier centerlines in one or both directions, but any segment with a passing sight distance less than 1,000 ft should also be considered as a no-passing zone.

Passing in the opposing lane of flow may be necessary on a two-lane highway. It is the only way to fill gaps forming in front of slow-moving vehicles in the traffic stream. Restrictions on the ability to pass significantly increase the rate at which platoons form in the traffic stream, since motorists are unable to pass slower vehicles in front of them.

Basic Speed–Flow Relationships

PTSF is the average percentage of time that vehicles must travel in platoons behind slower vehicles due to the inability to pass.

Exhibit 15-2 shows the relationships among flow rate, ATS, and PTSF for an extended directional segment of two-lane highway under base conditions. While the two directions of flow interact on a two-lane highway (because of passing maneuvers), this chapter analyzes each direction separately.

Exhibit 15-2(b) illustrates a critical characteristic of two-lane highways. Relatively low directional volumes create high PTSF values. With only 800 pc/h in one direction, PTSF ranges from 60% (with 200 pc/h opposing flow) to almost 80% (with 1,600 pc/h opposing flow). In contrast, typically acceptable speeds can be maintained on uninterrupted-flow multilane highways at relatively high proportions of capacity. However, on two-lane highways, service quality begins to deteriorate at relatively low demand flows.

CAPACITY AND LOS

Capacity

Capacity of a two-lane highway under base conditions is 1,700 pc/h in one direction, with a maximum of 3,200 pc/h in the two directions.

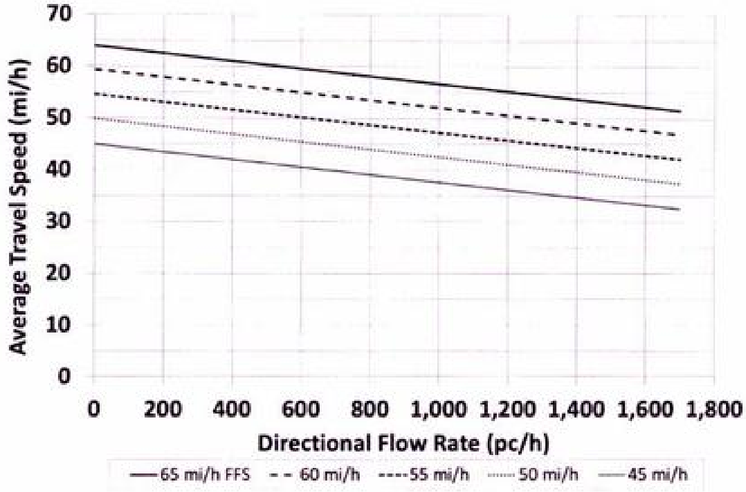
A two-lane highway's capacity under base conditions is 1,700 pc/h in one direction, with a limit of 3,200 pc/h for the total of both directions. Because of the interactions between directional flows, when a capacity of 1,700 pc/h is reached in one direction, the maximum opposing flow is limited to 1,500 pc/h.

Although capacity conditions are rarely observed in normal operation, they are important to consider for evacuation and special event planning.

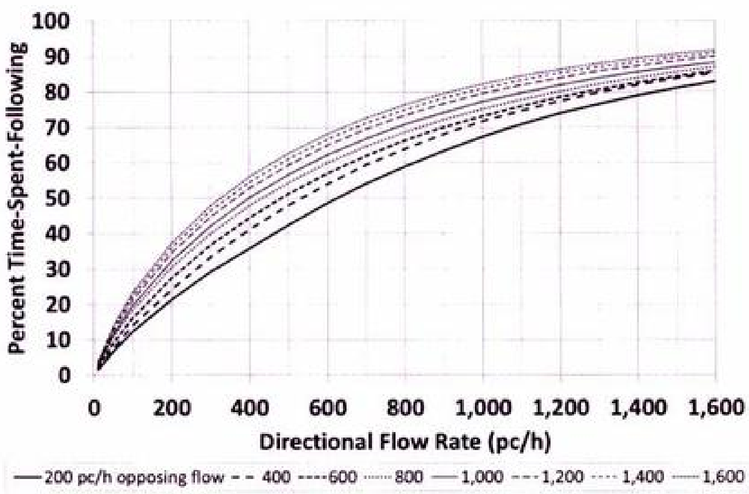
Capacity conditions are rarely observed except in short segments. Because service quality deteriorates at relatively low demand flow rates, most two-lane highways are upgraded before demand approaches capacity. Nevertheless, evaluating two-lane highway operations at capacity is important for evacuation planning, special event planning, and assessment of the downstream impacts of incident bottlenecks once they are cleared.

Two-way flow rates as high as 3,400 pc/h can be observed for short segments fed by high demands from multiple or multilane facilities. This may occur at tunnels or bridges, for example, but such flow rates cannot be expected over extended segments.

Exhibit 15-2
Speed–Flow and PTSF Relationships for Directional Segments with Base Conditions



(a) ATS Versus Directional Flow Rate



(b) PTSF Versus Directional Flow Rate

Capacity is not defined for bicycle facilities on two-lane highways because of a lack of data. Bicycle volumes approaching capacity do not often occur on two-lane highways except during special bicycle events, and little information is available on which to base a definition.

Levels of Service

Motorized Vehicle Mode

Because of the wide range of functions of two-lane highways, three service measures are used to describe motorized vehicle LOS, depending on the highway class:

1. *ATS* reflects mobility on a two-lane highway. It is defined as the highway segment's length divided by the average travel time for vehicles to traverse it during the analysis period.

2. *PTSF* represents the freedom to maneuver and the comfort and convenience of travel. It is the average percentage of time that vehicles must travel in platoons behind slower vehicles due to the inability to pass. Because this characteristic is difficult to measure in the field, a surrogate measure is the percentage of vehicles traveling at headways of less than 3.0 s at a representative location within the highway segment. PTSF also represents the approximate percentage of vehicles traveling in platoons.
3. *Percent of free-flow speed* (*PFFS*) represents the ability of vehicles to travel at or near the posted speed limit. It is the ratio of ATS to FFS. The exact relationship between FFS and speed limit depends heavily on local policies on setting such limits and on enforcement practices.

On Class I two-lane highways, both average speed and delay experienced while waiting to pass are important to motorists. Therefore, LOS is defined in terms of both ATS and PTSF for these highways. On Class II highways, travel speed is not a significant issue to drivers; therefore, LOS is defined in terms of PTSF only. On Class III highways, high speeds are not expected, and passing restrictions are not a major concern due to the relatively short lengths of Class III segments. Instead, motorists would like to make steady progress at or near the speed limit. Therefore, PFFS is used to define LOS on Class III highways. Exhibit 15-3 presents the motorized vehicle LOS criteria for two-lane highways.

Exhibit 15-3
Motorized Vehicle LOS for Two-Lane Highways

LOS	Class I Highways ATS (mi/h)	Class I Highways PTSF (%)	Class II Highways PTSF (%)	Class III Highways PFFS (%)
A	>55	≤35	≤40	>91.7
B	>50–55	>35–50	>40–55	>83.3–91.7
C	>45–50	>50–65	>55–70	>75.0–83.3
D	>40–45	>65–80	>70–85	>66.7–75.0
E	≤40	>80	>85	≤66.7
F	Demand exceeds capacity			

Note: For Class I highways, LOS is determined by the worse of ATS-based LOS and PTSF-based LOS.

Because driver expectations and operating characteristics on the three classes of two-lane highways are different, a single definition of operating conditions at each LOS is difficult to provide.

Two characteristics, however, have a significant impact on actual operations and driver perceptions of service:

- *Passing capacity*: Since passing maneuvers on two-lane highways are made in the opposing direction of flow, the ability to pass is limited by the opposing flow rate and by the distribution of gaps in the opposing flow.
- *Passing demand*: As platooning and PTSF increase in a given direction, the demand for passing maneuvers increases. As more drivers are caught in a platoon behind a slow-moving vehicle, they will desire to make more passing maneuvers.

Both passing capacity and passing demand are related to flow rates. As flow in the travel direction increases, passing demand in the travel direction also increases. As flow in the opposing direction increases, passing capacity in the travel direction decreases.

At LOS A, motorists experience high operating speeds on Class I highways and little difficulty in passing. Platoons of three or more vehicles are rare. On Class II highways, speed is controlled primarily by roadway conditions, but a small amount of platooning would be expected. On Class III highways, motorists can maintain operating speeds at or near the facility's FFS.

At LOS B, passing demand and passing capacity are balanced. On both Class I and Class II highways, the degree of platooning becomes noticeable. Some speed reductions are present on Class I highways. On Class III highways, maintenance of FFS operation becomes difficult, but the speed reduction is still relatively small.

At LOS C, most vehicles travel in platoons. Speeds are noticeably curtailed on all three classes of highway.

At LOS D, platooning increases significantly. Passing demand is high on both Class I and Class II facilities, but passing capacity approaches zero. A high percentage of vehicles travels in platoons, and PTSF is noticeable. On Class III highways, the fall-off from FFS is significant.

At LOS E, demand is approaching capacity. Passing on Class I and II highways is virtually impossible, and PTSF is more than 80%. Speeds are seriously curtailed. On Class III highways, speed is less than two-thirds the FFS. The lower limit of LOS E represents capacity.

LOS F exists whenever demand flow in one or both directions exceeds the segment's capacity. Operating conditions are unstable, and heavy congestion exists on all classes of two-lane highway.

Bicycle Mode

Bicycle LOS is based on a traveler perception model.

Bicycle levels of service for two-lane highway segments are based on a bicycle LOS (BLOS) score, which is in turn based on a traveler perception model. This score is based, in order of importance, on five variables:

- Average effective width of the outside through lane,
- Motorized vehicle volumes,
- Motorized vehicle speeds,
- Heavy vehicle (truck) volumes, and
- Pavement condition.

The LOS ranges for bicycles on two-lane and multilane highways are given in Exhibit 15-4.

Exhibit 15-4
Bicycle LOS for Two-Lane and Multilane Highways

LOS	BLOS Score
A	≤1.5
B	>1.5–2.5
C	>2.5–3.5
D	>3.5–4.5
E	>4.5–5.5
F	>5.5

3. CORE MOTORIZED VEHICLE METHODOLOGY

SCOPE OF THE METHODOLOGY

This chapter's methodology addresses the analysis of directional two-lane highway segments in general terrain (level or rolling) and directional segments on specific grades. All segments in mountainous terrain and all grades of 3% or more that cover a length of 0.6 mi or more must be analyzed as specific grades.

The extensions to the methodology presented in Section 4 address the analysis of passing and truck climbing lanes on directional segments. Section 4 also addresses specialized treatments for two-lane highways that cannot be evaluated with the core methodology.

The methodology is most directly used to determine the LOS on a uniform directional segment of two-lane highway by estimating the service measures that define LOS (ATS, PTSF, PFFS). Such an analysis can also be used to determine the capacity of the directional segment or the service flow rate that can be accommodated at any given LOS.

Spatial and Temporal Limits

This chapter's methodology applies to uniform directional segments of two-lane highway. While the two directions of flow interact through passing maneuvers (and limitations on passing maneuvers), each direction must be analyzed separately. Directional segments should have the same or similar traffic and roadway conditions in the direction being studied. Segment boundaries should be established at points where a change occurs in any of the following in the study direction: terrain, lane widths, shoulder width, facility classification, or demand flow rate. An analysis segment can contain no more than one passing or climbing lane in the study direction.

The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This method produces the following performance measures:

- ATS;
- Ratio of ATS to FFS;
- PTFS;
- LOS based on one or more of the above measures, depending on the highway class;
- Average travel time;
- v/c ratio;
- Total VMT during the analysis period; and
- Total VHT during the analysis period.

Strengths of the Methodology

The methodology provides a straightforward way to analyze uninterrupted-flow segments of two-lane highways and produces several useful performance measures as outputs. Extensions to the method in Section 4 allow the effects of passing and climbing lanes on two-lane highway operation to be evaluated.

Limitations of the Methodology

The methodology does not address two-lane highways with signalized intersections or other types of intersections requiring traffic on the highway to stop or yield. Isolated intersections on two-lane highways may be evaluated with the intersection methodologies given in Volume 3. Two-lane highways in urban and suburban areas with multiple signalized intersections spaced 2 mi or less apart should be analyzed as urban streets or arterials by using Chapter 17, Urban Street Segments. Operations of two-lane highways with signalized intersections closer than 2 mi apart are dominated by issues of signal progression and other arterial factors.

Even isolated intersections can have a significant effect on two-lane highway operations where queuing on the two-lane highway approaches is significant, particularly in cases where the intersection approach fails for any period of time, that is, has a v/c ratio > 1.00.

Alternative Tool Considerations

No alternative deterministic tools are in common use for two-lane highway analysis. An objective of NCHRP Project 17-65, Improved Analysis of Two-Lane Highway Capacity and Operational Performance, ongoing at the time of writing, was to develop a new or updated simulation tool for two-lane highways (*4*).

One of the potentially useful features of two-lane highway simulation is the ability to model specific configurations of a series of no-passing zones, exclusive passing lanes, and access points, all of which are now described in general terms (e.g., percent no-passing zones) in this chapter. Network simulation tools can also include traffic control devices at specific points.

Additional performance measures can be obtained from simulation results. One example is *follower density,* which is defined in terms of the number of followers per mile per lane. This concept, which is discussed in more detail in Chapter 36, Concepts: Supplemental, has attracted increasing international interest. Some examples that illustrate potential uses of two-lane highway simulation are presented elsewhere (*5*).

REQUIRED DATA AND SOURCES

Exhibit 15-5 lists the information necessary to apply the motorized vehicle methodology and suggests potential sources for obtaining these data. It also suggests default values for use when segment-specific information is not available. The user is cautioned that every use of a default value instead of a field-measured, segment-specific value may make the analysis results more approximate and less related to the specific conditions that describe the highway.

HCM defaults should only be used when (*a*) field data cannot be collected and (*b*) locally derived defaults do not exist.

Exhibit 15-5
Required Input Data, Potential Data Sources, and Default Values for Two-Lane Highway Motorized Vehicle Analysis

Required Data and Units	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Highway class (I, II, III)	Determine from functional class, land use, motorist expectation	Must be provided
Lane width (ft)	Road inventory, aerial photo	12 ft
Shoulder width (ft)	Road inventory, aerial photo	6 ft
Access point density (both sides) (access points/mi)	Field data, aerial photo	Class I and II: 8/mi Class III: 16/mi
Terrain type (level, rolling, specific grade)	Design plans, analyst judgment	Must be provided
Percent no-passing zone[a] (%)	Road inventory, aerial photo	Level: 20% Rolling: 40% More extreme: 80%
Free-flow speed (mi/h)	Direct speed measurements, estimate from design speed or speed limit	Base free-flow speed: Speed limit + 10 mi/h (see discussion in text)
Passing lane length (mi)	Field data, road inventory, aerial photo	Must be provided
	Demand Data	
Hourly demand volume (veh/h)	Field data, modeling	Must be provided
Directional volume split (%)	Field data, modeling	Must be provided
Analysis period length (min)	Set by analyst	15 min (0.25 h)
Peak hour factor[b] (decimal)	Field data	0.88
Heavy vehicle percentage (%)	Field data	6%[c]

Notes: ***Bold italic*** indicates high sensitivity (>20% change) of service measure to the choice of default value.
Bold indicates moderate sensitivity (10%–20% change) of service measure to the choice of default value.
[a] Percent no-passing zone may be different in each direction.
[b] Moderate to high sensitivity of service measures for very low PHF values. See the discussion in the text. PHF is not required when peak 15-min demand volumes are provided.
[c] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.

Care should be taken in using default values. The service measure results are sensitive to some of the input data listed in Exhibit 15-5. For example, passing lane length and percentage of no-passing zones both change the service measure result by more than 20% when these inputs are varied over their normal range. In addition, the free-flow speed results in a 10%–20% change in ATS when it is varied over its normal range. A very low peak hour factor (PHF) value (0.60) results in a greater than 20% change in PTSF and a greater than 10% change in ATS compared with the results obtained for the default value for PHF; more typical PHFs vary the service measure results by less than 10%. All other inputs change the service measure result by less than 10% when they are varied over their normal range (*6*).

OVERVIEW OF THE METHODOLOGY

Exhibit 15-6 illustrates the minimum steps required for the core motorized vehicle methodology. Because the three classes of highways use different service measures, not all of the steps are required for computing LOS for a given highway class. In particular, the computational step for estimating ATS is only a requirement for Class I and III highways, and the step for estimating PTSF is only a requirement for Class I and II highways. The step for estimating PFFS is only a requirement for Class III highways. However, an analyst may choose to include the nonrequired steps whenever the analysis would benefit from the information provided by the additional performance measures.

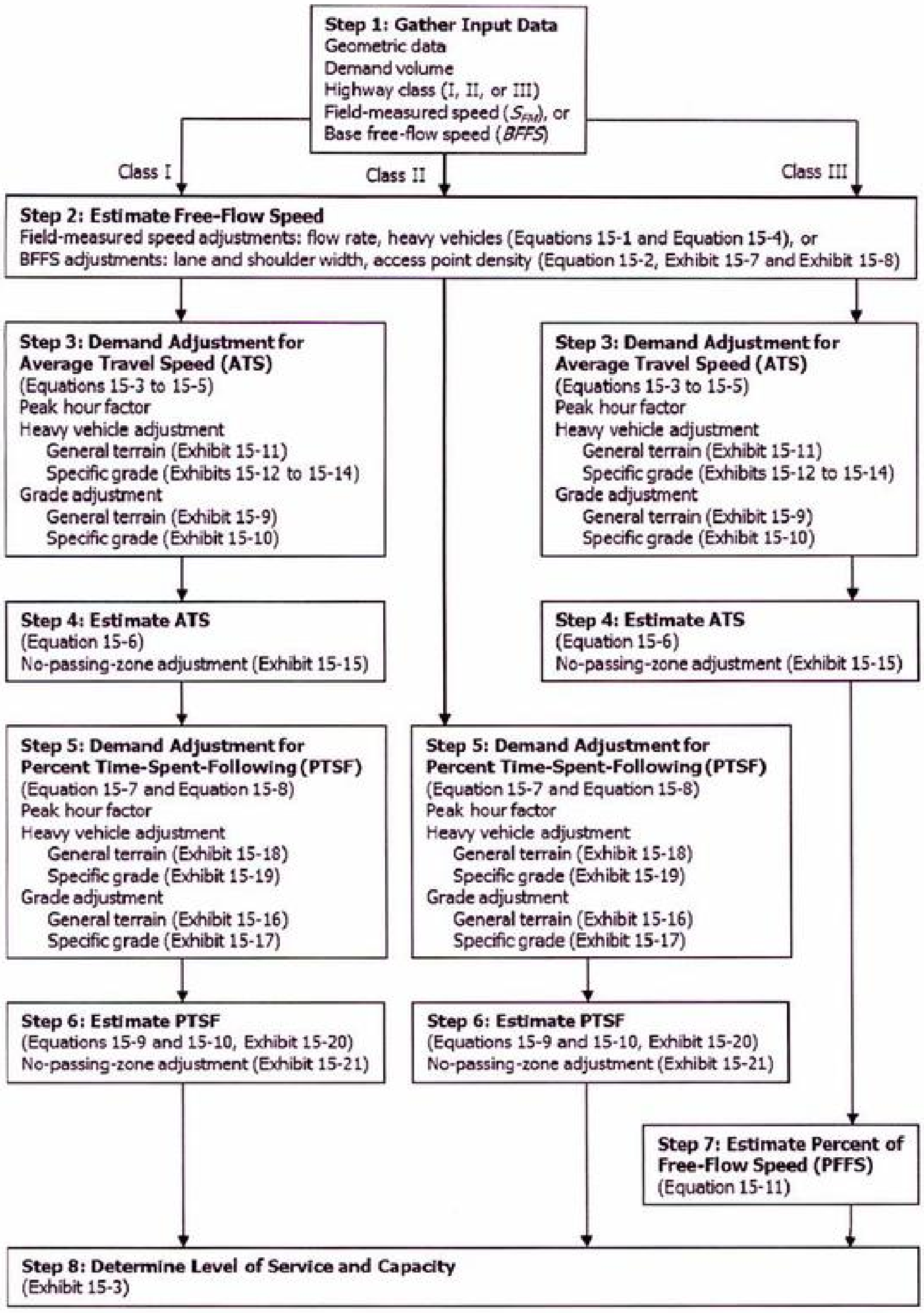


Exhibit 15-6
Flowchart of the Core Two-Lane Highway Methodology

COMPUTATIONAL STEPS

Step 1: Gather Input Data

Exhibit 15-5 listed the information that must be available before a two-lane highway segment can be analyzed and potential sources for that information. The exhibit suggested default values for use when neither site-specific data nor local default values are available.

Demand volumes are generally stated in vehicles per hour under prevailing conditions. They are converted in the methodology to demand flow rates in passenger cars per hour under base conditions. The PHF, in particular, is used to convert hourly volumes to flow rates.

If demand volumes are measured in 15-min increments, use of a PHF to convert volumes to flow rates is not necessary. The highest-volume 15-min period is selected, and the flow rate is simply calculated as this 15-min volume multiplied by 4. When this method is used, the PHF is set to 1.00.

In measuring demand volumes or flow rates, flow may be restricted by upstream bottlenecks or even signals that are more than 2 mi away from the study site (if they are closer, this chapter's methodology is not applicable). Downstream congestion may also affect flows in a study segment. Insofar as is possible, demand volumes and flow rates should reflect the situation that would exist with no upstream or downstream limiting factors.

Step 2: Estimate the FFS

There are three ways to estimate FFS on two-lane highways. FFS ranges from 45 to 70 mi/h on Class I and II highways.

A key step in the analysis of a two-lane highway is the determination of the FFS for the segment. There are three ways to estimate FFS.

Direct Field Measurement

Direct field measurement on the subject highway segment is preferred. Measurements should be taken only in the direction under analysis; if both directions are to be analyzed, separate measurements in each direction are made. Each directional measurement should be based on a sample of at least 100 vehicle speeds. The FFS can be directly measured as the mean speed under low-demand conditions (i.e., the two-way flow rate is less than or equal to 200 veh/h).

If the analysis segment cannot be directly observed, measurements from a similar facility (same highway class, same speed limit, similar environment, etc.) may be used.

Field Measurements at Higher Flow Rates

Sometimes, observation of total flow rates less than 200 veh/h may be difficult or impossible. In such cases, a speed sample may be taken at higher flow rates and adjusted accordingly. The same sampling approach is taken: each direction is separately observed, with each directional sample including at least 100 observed speeds. The measured mean speed is then adjusted with Equation 15-1:

Equation 15-1

$$FFS = S_{FM} + 0.00776\left(\frac{v}{f_{HV,ATS}}\right)$$

where

FFS = free-flow speed (mi/h);

S_{FM} = mean speed of sample ($v > 200$ veh/h) (mi/h);

v = total demand flow rate, both directions, during period of speed measurements (veh/h); and

$f_{HV,ATS}$ = heavy vehicle adjustment factor for ATS, from Equation 15-4 or Equation 15-5.

Estimating FFS

The FFS can be estimated indirectly if field data are not available. This is a greater challenge on two-lane highways than on other types of uninterrupted-flow facilities. FFS on Class I and II two-lane highways covers a significant range, from as low as 45 mi/h to as high as 70 mi/h. To estimate the FFS, the analyst must characterize the operating conditions of the facility in terms of a BFFS that reflects the nature of the traffic and the alignment of the facility. Unfortunately, because of the broad range of speeds that occur and the importance of local and regional factors influencing driver-desired speeds, little guidance on estimating the BFFS can be given.

Estimates of BFFS can be developed on the basis of speed data and local knowledge of operating conditions on similar facilities. As will be seen, once the BFFS is determined, adjustments for lane and shoulder widths and for the density of unsignalized access points are applied to estimate the FFS. In concept, the BFFS is the speed that would be expected on the basis of the facility's horizontal and vertical alignment if standard lane and shoulder widths were present and there were no roadside access points. Thus, the *design speed* of the facility might be an acceptable estimator of BFFS, since it is based primarily on horizontal and vertical alignment. Posted speed limits may not reflect current conditions or driver desires. A rough estimate of BFFS might be taken as the posted speed limit plus 10 mi/h.

Once a BFFS is determined, the FFS may be estimated as follows:

$$FFS = BFFS - f_{LS} - f_A$$

Equation 15-2

where

FFS = free-flow speed (mi/h),

$BFFS$ = base free-flow speed (mi/h),

f_{LS} = adjustment for lane and shoulder width (mi/h), and

f_A = adjustment for access point density (mi/h).

Adjustment factors for use in Equation 15-2 are found in Exhibit 15-7 (lane and shoulder width) and Exhibit 15-8 (access point density).

When field measurements are used to estimate FFS, standard approaches and sampling techniques should be applied. Guidance on field speed studies is provided in standard traffic engineering texts and elsewhere (3).

Exhibit 15-7
Adjustment Factor for Lane and Shoulder Width (f_{LS})

Lane Width (ft)	Shoulder Width (ft) ≥0, <2	≥2, <4	≥4, <6	≥6
≥9, <10	6.4	4.8	3.5	2.2
≥10, <11	5.3	3.7	2.4	1.1
≥11, <12	4.7	3.0	1.7	0.4
≥12	4.2	2.6	1.3	0.0

Exhibit 15-8
Adjustment Factor for Access Point Density (f_A)

Access Points per Mile (Both Sides)	Reduction in FFS (mi/h)
0	0.0
10	2.5
20	5.0
30	7.5
40	10.0

Note: Interpolation to the nearest 0.1 is recommended.

Estimating FFS will result in the same FFS in both directions. Field-measured FFS could be different in each direction.

The access point density is computed by dividing the total number of unsignalized intersections and driveways on *both* sides of the roadway segment by the length of the segment (in miles). Thus, in analyzing the two directions of the highway and estimating the FFS by using Equation 15-2, the FFS will be the same in *both* directions since the same access point density is being used. If the FFS is measured in the field, the value could be different in each direction.

The analyst may consider calculating a weighted-average FFS when the highway segment contains sharp horizontal curves with design speeds substantially below the rest of the segment.

If a highway contains sharp horizontal curves with design speeds substantially below those of the rest of the segment, determination of the FFS separately for curves and tangents and computation of a weighted-average FFS for the segment as a whole may be desirable.

The data for FFS relationships used in this chapter included both commuter and noncommuter traffic. There were no significant differences between the two. However, commuters and others who travel on the facility regularly are expected to use it more efficiently than recreational and other occasional users. If the effect of driver population is a concern, the FFS should be measured in the field.

Step 3: Demand Adjustment for ATS

This computational step is only required for Class I and Class III two-lane highways. LOS on Class II highways is not based on ATS; therefore, this step can be skipped for those highways, unless ATS or average travel time are additional desired performance measures from the analysis.

Demand volumes in both directions (analysis direction and opposing direction) must be converted to flow rates under equivalent base conditions with Equation 15-3:

Equation 15-3

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

where

$v_{i,ATS}$ = demand flow rate i for ATS estimation (pc/h);

i = "d" (analysis direction) or "o" (opposing direction);

V_i = demand volume for direction i (veh/h);

$f_{g,ATS}$ = grade adjustment factor, from Exhibit 15-9 or Exhibit 15-10; and

$f_{HV,ATS}$ = heavy vehicle adjustment factor, from Equation 15-4 or Equation 15-5.

PHF

The PHF represents the variation in traffic flow within the hour. Two-lane highway analysis is based on the demand flow rates for a peak 15-min period within the analysis hour—usually (but not necessarily) the peak hour.

ATS Grade Adjustment Factor

The grade adjustment factor $f_{g,ATS}$ depends on the terrain. Factors are defined for

- Extended segments (≥2 mi) of level terrain,
- Extended segments (≥2 mi) of rolling terrain,
- Specific upgrades, and
- Specific downgrades.

Any grade that is 3% or steeper and 0.6 mi or longer *must* be analyzed as a specific upgrade or downgrade, depending on the analysis direction being considered. However, a grade of 3% or more *may* be analyzed as a specific grade if it is 0.25 mi or longer.

Exhibit 15-9 shows grade adjustment factors for extended segments of level and rolling terrain, as well as for specific downgrades. Exhibit 15-9 is entered with the one-direction demand flow rate v_{vph}, in vehicles per hour.

Exhibit 15-9
ATS Grade Adjustment Factor ($f_{g,ATS}$) for Level Terrain, Rolling Terrain, and Specific Downgrades

One-Direction Demand Flow Rate, v_{vph} (veh/h)	Adjustment Factor	
	Level Terrain and Specific Downgrades	Rolling Terrain
≤100	1.00	0.67
200	1.00	0.75
300	1.00	0.83
400	1.00	0.90
500	1.00	0.95
600	1.00	0.97
700	1.00	0.98
800	1.00	0.99
≥900	1.00	1.00

Note: Interpolation to the nearest 0.01 is recommended.

If demand is expressed as an hourly volume, it must be divided by the PHF ($v_{vph} = V/PHF$) to obtain the appropriate factor. Other adjustment factor tables associated with Equation 15-3 are entered with this value as well.

Note that the adjustment factor for level terrain is 1.00, since level terrain is one of the base conditions. For the purposes of grade adjustment, specific downgrade segments are treated as level terrain.

Exhibit 15-10 shows grade adjustment factors for specific upgrades. The negative impact of upgrades on two-lane highway speeds increases as both the severity of the upgrade and its length increase. The impact declines as demand flow rate increases. At higher demand flow rates, lower speeds would already result, and the additional impact of the upgrades is less severe.

Exhibit 15-10
ATS Grade Adjustment Factor ($f_{g,ATS}$) for Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h) ≤100	200	300	400	500	600	700	800	≥900
≥3, <3.5	0.25	0.78	0.84	0.87	0.91	1.00	1.00	1.00	1.00	1.00
	0.50	0.75	0.83	0.86	0.90	1.00	1.00	1.00	1.00	1.00
	0.75	0.73	0.81	0.85	0.89	1.00	1.00	1.00	1.00	1.00
	1.00	0.73	0.79	0.83	0.88	1.00	1.00	1.00	1.00	1.00
	1.50	0.73	0.79	0.83	0.87	0.99	0.99	1.00	1.00	1.00
	2.00	0.73	0.79	0.82	0.86	0.98	0.98	0.99	1.00	1.00
	3.00	0.73	0.78	0.82	0.85	0.95	0.96	0.96	0.97	0.98
	≥4.00	0.73	0.78	0.81	0.85	0.94	0.94	0.95	0.95	0.96
≥3.5, <4.5	0.25	0.75	0.83	0.86	0.90	1.00	1.00	1.00	1.00	1.00
	0.50	0.72	0.80	0.84	0.88	1.00	1.00	1.00	1.00	1.00
	0.75	0.67	0.77	0.81	0.86	1.00	1.00	1.00	1.00	1.00
	1.00	0.65	0.73	0.77	0.81	0.94	0.95	0.97	1.00	1.00
	1.50	0.63	0.72	0.76	0.80	0.93	0.95	0.96	1.00	1.00
	2.00	0.62	0.70	0.74	0.79	0.93	0.94	0.96	1.00	1.00
	3.00	0.61	0.69	0.74	0.78	0.92	0.93	0.94	0.98	1.00
	≥4.00	0.61	0.69	0.73	0.78	0.91	0.91	0.92	0.96	1.00
≥4.5, <5.5	0.25	0.71	0.79	0.83	0.88	1.00	1.00	1.00	1.00	1.00
	0.50	0.60	0.70	0.74	0.79	0.94	0.95	0.97	1.00	1.00
	0.75	0.55	0.65	0.70	0.75	0.91	0.93	0.95	1.00	1.00
	1.00	0.54	0.64	0.69	0.74	0.91	0.93	0.95	1.00	1.00
	1.50	0.52	0.62	0.67	0.72	0.88	0.90	0.93	1.00	1.00
	2.00	0.51	0.61	0.66	0.71	0.87	0.89	0.92	0.99	1.00
	3.00	0.51	0.61	0.65	0.70	0.86	0.88	0.91	0.98	0.99
	≥4.00	0.51	0.60	0.65	0.69	0.84	0.86	0.88	0.95	0.97
≥5.5, <6.5	0.25	0.57	0.68	0.72	0.77	0.93	0.94	0.96	1.00	1.00
	0.50	0.52	0.62	0.66	0.71	0.87	0.90	0.92	1.00	1.00
	0.75	0.49	0.57	0.62	0.68	0.85	0.88	0.90	1.00	1.00
	1.00	0.46	0.56	0.60	0.65	0.82	0.85	0.88	1.00	1.00
	1.50	0.44	0.54	0.59	0.64	0.81	0.84	0.87	0.98	1.00
	2.00	0.43	0.53	0.58	0.63	0.81	0.83	0.86	0.97	0.99
	3.00	0.41	0.51	0.56	0.61	0.79	0.82	0.85	0.97	0.99
	≥4.00	0.40	0.50	0.55	0.61	0.79	0.82	0.85	0.97	0.99
≥6.5	0.25	0.54	0.64	0.68	0.73	0.88	0.90	0.92	1.00	1.00
	0.50	0.43	0.53	0.57	0.62	0.79	0.82	0.85	0.98	1.00
	0.75	0.39	0.49	0.54	0.59	0.77	0.80	0.83	0.96	1.00
	1.00	0.37	0.45	0.50	0.54	0.74	0.77	0.81	0.96	1.00
	1.50	0.35	0.45	0.49	0.54	0.71	0.75	0.79	0.96	1.00
	2.00	0.34	0.44	0.48	0.53	0.71	0.74	0.78	0.94	0.99
	3.00	0.34	0.44	0.48	0.53	0.70	0.73	0.77	0.93	0.98
	≥4.00	0.33	0.43	0.47	0.52	0.70	0.73	0.77	0.91	0.95

Note: Straight-line interpolation of $f_{g,ATS}$ for length of grade and demand flow permitted to the nearest 0.01.

ATS Heavy Vehicle Adjustment Factor

The base conditions for two-lane highways include 100% passenger cars in the traffic stream. This is a rare occurrence, and the presence of heavy vehicles in the traffic stream reduces the ATS.

Determining the heavy vehicle adjustment factor is a two-step process:

1. Passenger car equivalents are found for trucks (E_T) and recreational vehicles (RVs) (E_R) under prevailing conditions.
2. A heavy vehicle adjustment factor is computed from the passenger car equivalents with Equation 15-4:

Equation 15-4

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where

$f_{HV,ATS}$ = heavy vehicle adjustment factor for ATS estimation,

P_T = proportion of trucks in the traffic stream (decimal),

P_R = proportion of RVs in the traffic stream (decimal),

E_T = passenger car equivalent for trucks, and

E_R = passenger car equivalent for RVs.

The passenger car equivalent is the number of passenger cars displaced from the traffic stream by one truck or RV. Passenger car equivalents are defined for several situations:

- Extended sections of general level or rolling terrain,
- Specific upgrades, and
- Specific downgrades.

Exhibit 15-11 contains passenger car equivalents for trucks and RVs in general terrain segments and for specific downgrades, which are treated as level terrain in most cases. A special procedure is provided below to evaluate specific downgrades on which significant numbers of trucks must reduce their speed to crawl speed to maintain control.

Exhibit 15-11
ATS Passenger Car Equivalents for Trucks (E_T) and RVs (E_R) for Level Terrain, Rolling Terrain, and Specific Downgrades

Vehicle Type	Directional Demand Flow Rate, v_{vph} (veh/h)	Level Terrain and Specific Downgrades	Rolling Terrain
Trucks, E_T	≤100	1.9	2.7
	200	1.5	2.3
	300	1.4	2.1
	400	1.3	2.0
	500	1.2	1.8
	600	1.1	1.7
	700	1.1	1.6
	800	1.1	1.4
	≥900	1.0	1.3
RVs, E_R	All flows	1.0	1.1

Note: Interpolation to the nearest 0.1 is recommended.

Exhibit 15-12 and Exhibit 15-13 show passenger car equivalents for trucks and RVs, respectively, on specific upgrades.

Exhibit 15-12
ATS Passenger Car Equivalents for Trucks (E_T) on Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h)								
		≤100	200	300	400	500	600	700	800	≥900
≥3, <3.5	0.25	2.6	2.4	2.3	2.2	1.8	1.8	1.7	1.3	1.1
	0.50	3.7	3.4	3.3	3.2	2.7	2.6	2.6	1.9	1.6
	0.75	4.6	4.4	4.3	4.2	3.7	3.6	3.4	2.4	1.9
	1.00	5.2	5.0	4.9	4.9	4.4	4.2	4.1	3.0	2.3
	1.50	6.2	6.0	5.9	5.8	5.3	5.0	4.8	3.6	2.9
	2.00	7.3	6.9	6.7	6.5	5.7	5.5	5.3	4.1	3.5
	3.00	8.4	8.0	7.7	7.5	6.5	6.2	6.0	4.6	3.9
	≥4.00	9.4	8.8	8.6	8.3	7.2	6.9	6.6	4.8	3.7
≥3.5, <4.5	0.25	3.8	3.4	3.2	3.0	2.3	2.2	2.2	1.7	1.5
	0.50	5.5	5.3	5.1	5.0	4.4	4.2	4.0	2.8	2.2
	0.75	6.5	6.4	6.5	6.5	6.3	5.9	5.6	3.6	2.6
	1.00	7.9	7.6	7.4	7.3	6.7	6.6	6.4	5.3	4.7
	1.50	9.6	9.2	9.0	8.9	8.1	7.9	7.7	6.5	5.9
	2.00	10.3	10.1	10.0	9.9	9.4	9.1	8.9	7.4	6.7
	3.00	11.4	11.3	11.2	11.2	10.7	10.3	10.0	8.0	7.0
	≥4.00	12.4	12.2	12.2	12.1	11.5	11.2	10.8	8.6	7.5
≥4.5, <5.5	0.25	4.4	4.0	3.7	3.5	2.7	2.7	2.7	2.6	2.5
	0.50	6.0	6.0	6.0	6.0	5.9	5.7	5.6	4.6	4.2
	0.75	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	1.00	9.2	9.2	9.1	9.1	9.0	9.0	9.0	8.9	8.8
	1.50	10.6	10.6	10.6	10.6	10.5	10.4	10.4	10.2	10.1
	2.00	11.8	11.8	11.8	11.8	11.6	11.6	11.5	11.1	10.9
	3.00	13.7	13.7	13.6	13.6	13.3	13.1	13.0	11.9	11.3
	≥4.00	15.3	15.3	15.2	15.2	14.6	14.2	13.8	11.3	10.0
≥5.5, <6.5	0.25	4.8	4.6	4.5	4.4	4.0	3.9	3.8	3.2	2.9
	0.50	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2	7.2
	0.75	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1	9.1
	1.00	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.2	10.1
	1.50	11.9	11.9	11.9	11.9	11.8	11.8	11.8	11.7	11.6
	2.00	12.8	12.8	12.8	12.8	12.7	12.7	12.7	12.6	12.5
	3.00	14.4	14.4	14.4	14.4	14.3	14.3	14.3	14.2	14.1
	≥4.00	15.4	15.4	15.3	15.3	15.2	15.1	15.1	14.9	14.8
≥6.5	0.25	5.1	5.1	5.0	5.0	4.8	4.7	4.7	4.5	4.4
	0.50	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8
	0.75	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8	9.8
	1.00	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.3	10.2
	1.50	12.0	12.0	12.0	12.0	11.9	11.9	11.9	11.8	11.7
	2.00	12.9	12.9	12.9	12.9	12.8	12.8	12.8	12.7	12.6
	3.00	14.5	14.5	14.5	14.5	14.4	14.4	14.4	14.3	14.2
	≥4.00	15.4	15.4	15.4	15.4	15.3	15.3	15.3	15.2	15.1

Note: Interpolation for length of grade and demand flow rate to the nearest 0.1 is recommended.

Exhibit 15-13
ATS Passenger Car Equivalents for RVs (E_R) on Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h)								
		≤100	200	300	400	500	600	700	800	≥900
≥3, <3.5	≤0.25	1.1	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0
	>0.25, ≤0.75	1.2	1.2	1.1	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75, ≤1.25	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>1.25, ≤2.25	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>2.25	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
≥3.5, <4.5	≤0.75	1.3	1.2	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75, ≤3.50	1.4	1.3	1.2	1.1	1.0	1.0	1.0	1.0	1.0
	>3.50	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
≥4.5, <5.5	≤2.50	1.5	1.4	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
≥5.5, <6.5	≤0.75	1.5	1.4	1.3	1.1	1.0	1.0	1.0	1.0	1.0
	>0.75, ≤2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50, ≤3.50	1.6	1.5	1.4	1.3	1.2	1.1	1.0	1.0	1.0
	>3.50	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.2	1.1
≥6.5	≤2.50	1.6	1.5	1.4	1.2	1.0	1.0	1.0	1.0	1.0
	>2.50, ≤3.50	1.6	1.5	1.4	1.2	1.3	1.3	1.3	1.3	1.3
	>3.50	1.6	1.6	1.6	1.5	1.5	1.5	1.4	1.4	1.4

Note: Interpolation in this exhibit is not recommended.

ATS Passenger Car Equivalents for Specific Downgrades Where Trucks Travel at Crawl Speed

As noted previously, any downgrade of 3% or more *and* 0.6 mi or longer must be analyzed as a specific downgrade. If the slope of the downgrade varies, it should be analyzed as a single composite by using an average grade computed by dividing the total change in elevation by the total length of grade and expressing the result as a percentage.

Most specific downgrades will be treated as level terrain for analysis purposes. However, some downgrades are severe enough to force some trucks into crawl speed. In such cases, the truck drivers are forced to operate in a low gear to apply engine braking, since the normal brake system would not be sufficient to slow or stop a heavy vehicle from gaining too much momentum as it travels down a sharp downgrade. There are no general guidelines for identifying when or where these situations will occur other than direct observation of heavy vehicle operations.

When this situation exists, the heavy vehicle adjustment factor $f_{HV,ATS}$ is found with Equation 15-5 instead of Equation 15-4:

$$f_{HV,ATS} = \frac{1}{1 + P_{TC} \times P_T(E_{TC} - 1) + (1 - P_{TC}) \times P_T \times (E_T - 1) + P_R(E_R - 1)}$$

Equation 15-5

where

P_{TC} = proportion of trucks operating at crawl speed (decimal); and

E_{TC} = passenger car equivalent for trucks operating at crawl speed, from Exhibit 15-14.

All other variables are as previously defined. Note that P_{TC} is the flow rate of trucks traveling at crawl speed divided by the flow rate of all trucks.

Exhibit 15-14
ATS Passenger Car Equivalents (E_{TC}) for Trucks on Downgrades Traveling at Crawl Speed

Difference Between FFS and Truck Crawl Speed (mi/h)	Directional Demand Flow Rate, v_{vph} (veh/h)								
	≤100	200	300	400	500	600	700	800	≥900
≤15	4.7	4.1	3.6	3.1	2.6	2.1	1.6	1.0	1.0
20	9.9	8.7	7.8	6.7	5.8	4.9	4.0	2.7	1.0
25	15.1	13.5	12.0	10.4	9.0	7.7	6.4	5.1	3.8
30	22.0	19.8	17.5	15.6	13.1	11.6	9.2	6.1	4.1
35	29.0	26.0	23.1	20.1	17.3	14.6	11.9	9.2	6.5
≥40	35.9	32.3	28.6	24.9	21.4	18.1	14.7	11.3	7.9

Note: Interpolation against both speed difference and demand flow rate to the nearest 0.1 is recommended.

Step 4: Estimate the ATS

As was the case with Step 3, this step is only required for Class I and Class III two-lane highways. Class II highways do not use ATS as a LOS measure, but this step can be applied if ATS or average travel time is a desired outcome of the analysis of a Class II highway.

The ATS is estimated from the FFS, the demand flow rate, the opposing flow rate, and the percentage of no-passing zones in the analysis direction. The ATS is computed from Equation 15-6:

$$ATS_d = FFS - 0.00776(v_{d,ATS} + v_{o,ATS}) - f_{np,ATS}$$

Equation 15-6

where

ATS_d = average travel speed in the analysis direction (mi/h);

FFS = free-flow speed (mi/h);

$v_{d,ATS}$ = demand flow rate for ATS determination in the analysis direction (pc/h);

$v_{o,ATS}$ = demand flow rate for ATS determination in the opposing direction (pc/h); and

$f_{np,ATS}$ = adjustment factor for ATS determination for the percentage of no-passing zones in the analysis direction, from Exhibit 15-15.

Exhibit 15-15
ATS Adjustment Factor for No-Passing Zones ($f_{np,ATS}$)

Opposing Demand Flow Rate, v_o (pc/h)	Percent No-Passing Zones ≤20	40	60	80	100
		FFS ≥ 65 mi/h			
≤100	1.1	2.2	2.8	3.0	3.1
200	2.2	3.3	3.9	4.0	4.2
400	1.6	2.3	2.7	2.8	2.9
600	1.4	1.5	1.7	1.9	2.0
800	0.7	1.0	1.2	1.4	1.5
1,000	0.6	0.8	1.1	1.1	1.2
1,200	0.6	0.8	0.9	1.0	1.1
1,400	0.6	0.7	0.9	0.9	0.9
≥1,600	0.6	0.7	0.7	0.7	0.8
		FFS = 60 mi/h			
≤100	0.7	1.7	2.5	2.8	2.9
200	1.9	2.9	3.7	4.0	4.2
400	1.4	2.0	2.5	2.7	3.9
600	1.1	1.3	1.6	1.9	2.0
800	0.6	0.9	1.1	1.3	1.4
1,000	0.6	0.7	0.9	1.1	1.2
1,200	0.5	0.7	0.9	0.9	1.1
1,400	0.5	0.6	0.8	0.8	0.9
≥1,600	0.5	0.6	0.7	0.7	0.7
		FFS = 55 mi/h			
≤100	0.5	1.2	2.2	2.6	2.7
200	1.5	2.4	3.5	3.9	4.1
400	1.3	1.9	2.4	2.7	2.8
600	0.9	1.1	1.6	1.8	1.9
800	0.5	0.7	1.1	1.2	1.4
1,000	0.5	0.6	0.8	0.9	1.1
1,200	0.5	0.6	0.7	0.9	1.0
1,400	0.5	0.6	0.7	0.7	0.9
≥1,600	0.5	0.6	0.6	0.6	0.7
		FFS = 50 mi/h			
≤100	0.2	0.7	1.9	2.4	2.5
200	1.2	2.0	3.3	3.9	4.0
400	1.1	1.6	2.2	2.6	2.7
600	0.6	0.9	1.4	1.7	1.9
800	0.4	0.6	0.9	1.2	1.3
1,000	0.4	0.4	0.7	0.9	1.1
1,200	0.4	0.4	0.7	0.8	1.0
1,400	0.4	0.4	0.6	0.7	0.8
≥1,600	0.4	0.4	0.5	0.5	0.5
		FFS ≤ 45 mi/h			
≤100	0.1	0.4	1.7	2.2	2.4
200	0.9	1.6	3.1	3.8	4.0
400	0.9	0.5	2.0	2.5	2.7
600	0.4	0.3	1.3	1.7	1.8
800	0.3	0.3	0.8	1.1	1.2
1,000	0.3	0.3	0.6	0.8	1.1
1,200	0.3	0.3	0.6	0.7	1.0
1,400	0.3	0.3	0.6	0.6	0.7
≥1,600	0.3	0.3	0.4	0.4	0.6

Note: Interpolation of $f_{np,ATS}$ for percent no-passing zones, demand flow rate, and FFS to the nearest 0.1 is recommended.

Exhibit 15-15 is entered with v_o in passenger cars per hour, not v_{vph} in vehicles per hour. Demand flow rates were determined by Equation 15-3 and are used in the determination of *ATS*. As shown in this exhibit, the effect of no-passing zones is greatest when opposing flow rates are low. As opposing flow rates increase, the effect decreases to zero, since passing and no-passing zones become irrelevant when the opposing flow rate allows no opportunities to pass.

Step 5: Demand Adjustment for PTSF

This computational step is applied only in cases of Class I and Class II two-lane highways. LOS on Class III highways is not based on PTSF, and therefore this step can be skipped for those highways, unless PTSF is a desired output performance measure.

The demand volume adjustment process for estimating PTSF is structurally similar to that for ATS. The general approach is the same, but different adjustment factors are used, and the resulting adjusted flow rates are different from those used in estimating ATS. Therefore, a detailed discussion of the process is not provided, since it is the same as described above for ATS estimates.

Equation 15-7 and Equation 15-8 are used to determine demand flow rates for the estimation of PTSF:

$$v_{i,PTSF} = \frac{V_i}{PHF \times f_{g,PTSF} \times f_{HV,PTSF}}$$

Equation 15-7

$$f_{HV,PTSF} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Equation 15-8

where

$v_{i,PTSF}$ = demand flow rate *i* for determination of PTSF (pc/h);

i = "d" (analysis direction) or "o" (opposing direction);

$f_{g,PTSF}$ = grade adjustment factor for PTSF determination, from Exhibit 15-16 or Exhibit 15-17; and

$f_{HV,PTSF}$ = heavy vehicle adjustment factor for PTSF determination.

All other variables are as previously defined.

PTSF Grade Adjustment Factor

As was the case for the ATS adjustment process, grade adjustment factors are defined for general terrain segments (level or rolling), specific upgrades, and specific downgrades. Exhibit 15-16 gives the adjustment factors for general terrain segments and specific downgrades (which are treated as level terrain). Exhibit 15-17 shows adjustment factors for specific upgrades. These adjustments are used to compute demand flow rates, and the exhibits are again entered with $v_{vph} = V/PHF$.

Exhibit 15-16
PTSF Grade Adjustment Factor ($f_{g,PTSF}$) for Level Terrain, Rolling Terrain, and Specific Downgrades

Directional Demand Flow Rate, v_{vph} (veh/h)	Level Terrain and Specific Downgrades	Rolling Terrain
≤100	1.00	0.73
200	1.00	0.80
300	1.00	0.85
400	1.00	0.90
500	1.00	0.96
600	1.00	0.97
700	1.00	0.99
800	1.00	1.00
≥900	1.00	1.00

Note: Interpolation to the nearest 0.01 is recommended.

Exhibit 15-17
PTSF Grade Adjustment Factor ($f_{g,PTSF}$) for Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h)								
		≤100	200	300	400	500	600	700	800	≥900
≥3, <3.5	0.25	1.00	0.99	0.97	0.96	0.92	0.92	0.92	0.92	0.92
	0.50	1.00	0.99	0.98	0.97	0.93	0.93	0.93	0.93	0.93
	0.75	1.00	0.99	0.98	0.97	0.93	0.93	0.93	0.93	0.93
	1.00	1.00	0.99	0.98	0.97	0.93	0.93	0.93	0.93	0.93
	1.50	1.00	0.99	0.98	0.97	0.94	0.94	0.94	0.94	0.94
	2.00	1.00	0.99	0.98	0.98	0.95	0.95	0.95	0.95	0.95
	3.00	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.96	0.96
	≥4.00	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.97	0.97
≥3.5, <4.5	0.25	1.00	0.99	0.98	0.97	0.94	0.93	0.93	0.92	0.92
	0.50	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.96	0.95
	0.75	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.96	0.96
	1.00	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.97	0.97
	1.50	1.00	1.00	0.99	0.99	0.97	0.97	0.97	0.97	0.97
	2.00	1.00	1.00	0.99	0.99	0.98	0.98	0.98	0.98	0.98
	3.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	≥4.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
≥4.5, <5.5	0.25	1.00	1.00	1.00	1.00	1.00	0.99	0.99	0.97	0.97
	≥0.50	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
≥5.5	All	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

Note: Interpolation for length of grade and demand flow rate to the nearest 0.01 is recommended.

PTSF Heavy Vehicle Adjustment Factor

The process for determining the heavy vehicle adjustment factor used in estimating PTSF (Equation 15-8) is similar to that used in estimating ATS. Passenger car equivalents must be found for trucks (E_T) and RVs (E_R).

Equivalents for both trucks and RVs in general terrain segments (level, rolling) and on specific downgrades (which are treated as level terrain) are found in Exhibit 15-18. In estimating PTSF, there is no special procedure for trucks traveling at crawl speed on specific downgrades. Equivalents for trucks and RVs on specific upgrades are found in Exhibit 15-19.

Exhibit 15-18
PTSF Passenger Car Equivalents for Trucks (E_T) and RVs (E_R) for Level Terrain, Rolling Terrain, and Specific Downgrades

Vehicle Type	Directional Demand Flow Rate, v_{vph} (veh/h)	Level and Specific Downgrade	Rolling
Trucks, E_T	≤100	1.1	1.9
	200	1.1	1.8
	300	1.1	1.7
	400	1.1	1.6
	500	1.0	1.4
	600	1.0	1.2
	700	1.0	1.0
	800	1.0	1.0
	≥900	1.0	1.0
RVs, E_R	All	1.0	1.0

Note: Interpolation in this exhibit is not recommended.

Exhibit 15-19
PTSF Passenger Car Equivalents for Trucks (E_T) and RVs (E_R) on Specific Upgrades

Grade (%)	Grade Length (mi)	Directional Demand Flow Rate, v_{vph} (veh/h) ≤100	200	300	400	500	600	700	800	≥900
		Passenger Car Equivalents for Trucks (E_T)								
≥3, <3.5	≤2.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.5	1.3	1.3	1.2	1.0	1.0	1.0	1.0	1.0
	≥4.00	1.6	1.4	1.3	1.3	1.0	1.0	1.0	1.0	1.0
≥3.5, <4.5	≤1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	2.00	1.6	1.3	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	3.00	1.8	1.4	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	≥4.00	2.1	1.9	1.8	1.7	1.4	1.4	1.4	1.4	1.4
≥4.5, <5.5	≤1.00	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.50	1.1	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2
	2.00	1.7	1.6	1.6	1.6	1.5	1.4	1.4	1.3	1.3
	3.00	2.4	2.2	2.2	2.1	1.9	1.8	1.8	1.7	1.7
	≥4.00	3.5	3.1	2.9	2.7	2.1	2.0	2.0	1.8	1.8
≥5.5, <6.5	≤0.75	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	1.00	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2
	1.50	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.6
	2.00	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.8	1.8
	3.00	3.4	3.2	3.0	2.9	2.4	2.3	2.3	1.9	1.9
	≥4.00	4.5	4.1	3.9	3.7	2.9	2.7	2.6	2.0	2.0
≥6.5	≤0.50	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
	0.75	1.0	1.0	1.0	1.0	1.1	1.1	1.1	1.0	1.0
	1.00	1.3	1.3	1.3	1.4	1.4	1.5	1.5	1.4	1.4
	1.50	2.1	2.1	2.1	2.1	2.0	2.0	2.0	2.0	2.0
	2.00	2.9	2.8	2.7	2.7	2.4	2.4	2.3	2.3	2.3
	3.00	4.2	3.9	3.7	3.6	3.0	2.8	2.7	2.2	2.2
	≥4.00	5.0	4.6	4.4	4.2	3.3	3.1	2.9	2.7	2.5
		Passenger Car Equivalents for RVs (E_R)								
All	All	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0

Note: Interpolation for length of grade and demand flow rate to the nearest 0.1 is recommended.

Step 6: Estimate the PTSF

This step is only required for Class I and Class II two-lane highways. Class III highways do not use PTSF to determine LOS, but users may apply this step if PTSF is a desired output of the analysis.

Once the demand flows for estimating PTSF are computed, the PTSF is estimated with Equation 15-9:

Equation 15-9

$$PTSF_d = BPTSF_d + f_{np,PTSF}\left(\frac{v_{d,PTSF}}{v_{d,PTSF} + v_{o,PTSF}}\right)$$

where

$PTSF_d$ = percent time-spent-following in the analysis direction (decimal);

$BPTSF_d$ = base percent time-spent-following in the analysis direction, from Equation 15-10;

$f_{np,PTSF}$ = adjustment to PTSF for the percentage of no-passing zones in the analysis segment;

$v_{d,PTSF}$ = demand flow rate in the analysis direction for estimation of PTSF (pc/h); and

$v_{o,PTSF}$ = demand flow rate in the opposing direction for estimation of PTSF (pc/h).

The base percent time-spent-following (BPTSF) applies to base conditions and is estimated by Equation 15-10:

Equation 15-10

$$BPTSF_d = 100\left[1 - \exp\left(a v_d^b\right)\right]$$

where a and b are constants drawn from Exhibit 15-20 and all other terms are as previously defined.

Exhibit 15-21 provides values of the no-passing-zone adjustment factor $f_{np,PTSF}$. Both Exhibit 15-20 and Exhibit 15-21 are entered with demand flow rates fully converted to passenger cars per hour under base conditions (v_o and v_d).

Exhibit 15-20
PTSF Coefficients for Use in Equation 15-10 for Estimating BPTSF

Opposing Demand Flow Rate, v_o (pc/h)	Coefficient a	Coefficient b
≤200	-0.0014	0.973
400	-0.0022	0.923
600	-0.0033	0.870
800	-0.0045	0.833
1,000	-0.0049	0.829
1,200	-0.0054	0.825
1,400	-0.0058	0.821
≥1,600	-0.0062	0.817

Note: Straight-line interpolation of a to the nearest 0.0001 and b to the nearest 0.001 is recommended.

Exhibit 15-21
No-Passing-Zone Adjustment Factor ($f_{np,PTSF}$) for Determination of PTSF

Total Two-Way Flow Rate, $v = v_d + v_o$ (pc/h)	Percent No-Passing Zones					
	0	**20**	**40**	**60**	**80**	**100**
Directional Split = 50/50						
≤200	9.0	29.2	43.4	49.4	51.0	52.6
400	16.2	41.0	54.2	61.6	63.8	65.8
600	15.8	38.2	47.8	53.2	55.2	56.8
800	15.8	33.8	40.4	44.0	44.8	46.6
1,400	12.8	20.0	23.8	26.2	27.4	28.6
2,000	10.0	13.6	15.8	17.4	18.2	18.8
2,600	5.5	7.7	8.7	9.5	10.1	10.3
3,200	3.3	4.7	5.1	5.5	5.7	6.1
Directional Split = 60/40						
≤200	11.0	30.6	41.0	51.2	52.3	53.5
400	14.6	36.1	44.8	53.4	55.0	56.3
600	14.8	36.9	44.0	51.1	52.8	54.6
800	13.6	28.2	33.4	38.6	39.9	41.3
1,400	11.8	18.9	22.1	25.4	26.4	27.3
2,000	9.1	13.5	15.6	16.0	16.8	17.3
2,600	5.9	7.7	8.6	9.6	10.0	10.2
Directional Split = 70/30						
≤200	9.9	28.1	38.0	47.8	48.5	49.0
400	10.6	30.3	38.6	46.7	47.7	48.8
600	10.9	30.9	37.5	43.9	45.4	47.0
800	10.3	23.6	28.4	33.3	34.5	35.5
1,400	8.0	14.6	17.7	20.8	21.6	22.3
2,000	7.3	9.7	11.7	13.3	14.0	14.5
Directional Split = 80/20						
≤200	8.9	27.1	37.1	47.0	47.4	47.9
400	6.6	26.1	34.5	42.7	43.5	44.1
600	4.0	24.5	31.3	38.1	39.1	40.0
800	3.8	18.5	23.5	28.4	29.1	29.9
1,400	3.5	10.3	13.3	16.3	16.9	32.2
2,000	3.5	7.0	8.5	10.1	10.4	10.7
Directional Split = 90/10						
≤200	4.6	24.1	33.6	43.1	43.4	43.6
400	0.0	20.2	28.3	36.3	36.7	37.0
600	-3.1	16.8	23.5	30.1	30.6	31.1
800	-2.8	10.5	15.2	19.9	20.3	20.8
1,400	-1.2	5.5	8.3	11.0	11.5	11.9

Note: Straight-line interpolation of $f_{np,PTSF}$ for percent no-passing zones, demand flow rate, and directional split is recommended to the nearest 0.1.

Note that in Exhibit 15-21, the adjustment factor depends on the total two-way demand flow rate, even though the factor is applied to a single directional analysis. The factor reflects not only the percent of no-passing zones in the analysis segment but also the directional distribution of traffic. The directional distribution measure is the same regardless of the direction being considered. Thus, for example, splits of 70/30 and 30/70 result in the same factor, all other variables being constant. However, Equation 15-9 adjusts the factor to reflect the balance of flows in the analysis and opposing directions.

Step 7: Estimate the PFFS

This step is only required for the analysis of Class III two-lane highways. PFFS is not used in the determination of LOS for Class I or Class II facilities, but this step can be performed if PFFS is a desired output performance measure from the analysis. The computation is straightforward, since both the FFS and the ATS have already been determined in previous steps. PFFS is estimated from Equation 15-11:

$$PFFS = \frac{ATS_d}{FFS}$$

Equation 15-11

where all terms are as previously defined.

Step 8: Determine LOS and Capacity

LOS Determination

At this point in the analysis, the values of any needed measure(s) have been determined. The LOS is found by comparing the appropriate measures with the criteria of Exhibit 15-3. The measure(s) used must be appropriate to the class of the facility being studied:

- Class I: ATS and PTSF,
- Class II: PTSF, and
- Class III: PFFS.

For Class I highways, two service measures are applied. When Exhibit 15-3 is entered, therefore, two LOS designations can be obtained. The worse of the two is the prevailing LOS. For example, if ATS results in a LOS C designation and PTSF results in a LOS D designation, LOS D is assigned.

Capacity Determination

Capacity, which exists at the boundary between LOS E and F, is not determined by a service measure. Under base conditions, the capacity of a two-lane highway (in one direction) is 1,700 pc/h. To determine the capacity under prevailing conditions, relevant adjustment factors must be applied to Equation 15-3 and Equation 15-7. In this case, however, the demand flow rate of 1,700 pc/h under base conditions is known, and the demand flow rate under prevailing conditions is sought.

First, capacity is defined as a flow rate, so the PHF in Equation 15-3 and Equation 15-7 is set at 1.00. Then, Equation 15-12 or Equation 15-13 (or both) are applied, as described below.

A v/c ratio is also a common performance measure of interest in LOS and capacity analysis. It is most easily computed for two-lane highways with Equation 15-18:

Equation 15-18

$$v/c = \frac{v_d}{1{,}700}$$

where v_d is the directional demand flow rate converted to equivalent base conditions.

The difficulty is that there may be two values of v_d: one for estimating ATS and another for estimating PTSF (depending on the class of highway). For Class I highways, where both measures are used, the result yielding the highest v/c ratio would be used. For Class II highways, only PTSF is used, and only one value would exist. For Class III highways, only ATS is used, and only one value would exist.

4. EXTENSIONS TO THE MOTORIZED VEHICLE METHODOLOGY

Adding an extra lane to a two-lane highway to improve passing opportunities improves the highway's operational performance and therefore may improve LOS. This section provides procedures for estimating the effects of passing and climbing lanes on two-lane highway performance. In addition, this section discusses the potential for geometric and operational treatments for improving traffic operations on two-lane highways. Chapter 26, Freeway and Highway Segments: Supplemental, provides a method for estimating the capacity of a work zone on a two-lane highway.

PASSING LANES

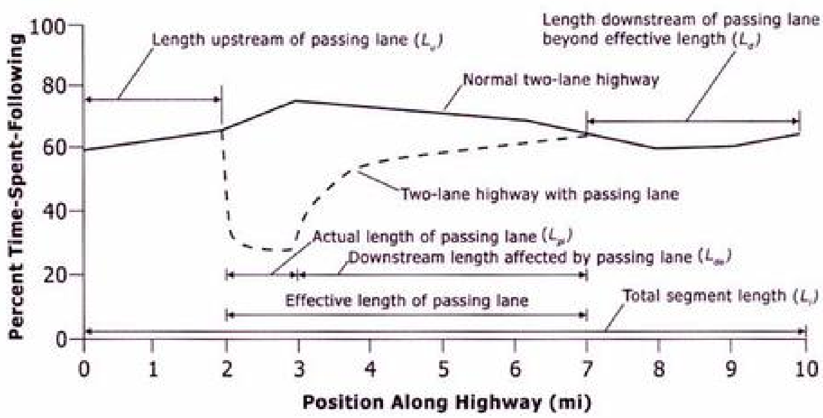
An added lane on a specific grade is considered to be a climbing lane and not a passing lane.

A *passing lane* is a lane added within a portion of the two-lane highway segment to improve passing opportunities in one direction of travel. For the purposes of this procedure, passing lanes only exist in level and rolling terrain. Added lanes on specific grades are considered to be *climbing lanes* and are addressed later in this section.

The effective length of a passing lane is longer than its actual length.

Exhibit 15-22 illustrates the operational effect of a passing lane on PTSF. It shows that the passing lane provides operational benefits for some distance downstream before PTSF returns to its former level (without a passing lane). Thus, a passing lane's effective length is greater than its actual length.

Exhibit 15-22
Operational Effect of a Passing Lane on PTSF



Source: Harwood and Hoban (7).

Exhibit 15-23 gives the length of the downstream segment affected by the passing lane for both ATS and PTSF. In the case of ATS, the effect is limited to 1.7 mi in all cases. Where PTSF is concerned, the effect can be far longer than the passing lane itself—up to 13 mi for low demand flow rates.

The analysis segment should include the entire length of the passing lane's downstream effect.

The procedure here is intended for the analysis of directional segments in level or rolling terrain that encompass the entire passing lane. Segments of the highway upstream and downstream of the passing lane may be included in the analysis. Inclusion of the full length of the passing lane's downstream effect in the analysis segment is recommended.

Exhibit 15-23
Downstream Length of Roadway Affected by Passing Lanes on Directional Segments in Level and Rolling Terrain

Directional Demand Flow Rate, v_d (pc/h)	Downstream Length of Roadway Affected, L_{de} (mi)	
	PTSF	ATS
≤200	13.0	1.7
300	11.6	1.7
400	8.1	1.7
500	7.3	1.7
600	6.5	1.7
700	5.7	1.7
800	5.0	1.7
900	4.3	1.7
≥1,000	3.6	1.7

Note: Interpolation to the nearest 0.1 is recommended.

Care should be taken in considering the effect of passing lanes on service flow rates; they are greatly affected by the length of the passing lane relative to the length of the analysis segment.

Because of the downstream effect on PTSF, the LOS on a two-lane highway segment that is determined by PTSF (Class I and Class II) may be significantly improved by the addition of a passing lane. However, care must be taken in considering the impact of a passing lane on service volumes or service flow rates. The result is highly dependent on the relative lengths of the analysis segment and the passing lane. If the analysis segment includes only the length of the passing lane and its downstream effective length (on PTSF), the passing lane may appear to increase service flow rates dramatically at LOS A–D. (Capacity, and therefore LOS E, would not be affected.) However, if additional lengths are included in the analysis segment, this impact is reduced, sometimes considerably. Thus, apparent increases in service volumes or service flow rates must be carefully considered in the context of how they were obtained.

The steps in this special analysis procedure are as follows.

Step 1: Conduct an Analysis Without the Passing Lane

The first step in the operational analysis of the impact of a passing lane is to follow the core methodology steps for a two-lane highway described in Section 3. The remainder of the procedure essentially predicts the improvement caused by the passing lane compared with a similar segment without a passing lane.

Step 2: Divide the Segment into Regions

The analysis segment can be divided into four regions, as follows:

1. Length upstream of the passing lane L_u (all lengths are in miles),
2. Length of the passing lane L_{pl},
3. Length downstream of the passing lane within its effective length L_{de}, and
4. Length downstream of the passing lane beyond its effective length L_d.

Some of these regions may not be involved in a particular analysis. Region 2, the passing lane, must be included in every analysis. In addition, it is strongly recommended, but not absolutely necessary, that Region 3 be included. Regions 1 and 4 are optional, and inclusion is at the discretion of the analyst.

The four lengths must add up to the total length of the analysis segment. The analysis regions and their lengths will differ for estimations of ATS and PTSF, since the downstream effects indicated in Exhibit 15-23 differ for each.

The length of the passing lane L_{pl} is either the length of the passing lane as constructed or the planned length. It should include the length of the lane addition as well as the length of the entrance and exit tapers. The procedure is calibrated for passing lanes within the optimal lengths shown in Exhibit 15-24. Passing lanes that are substantially shorter or longer than the optimums shown may provide less operational benefit than predicted by this procedure.

Exhibit 15-24
Optimal Lengths of Passing Lanes on Two-Lane Highways

Directional Demand Flow Rate, v_d (pc/h)	Optimal Passing Lane Length (mi)
≤100	≤0.50
>100, ≤400	>0.50, ≤0.75
>400, ≤700	>0.75, ≤1.00
≥700	>1.00, ≤2.00

The length of the conventional two-lane highway segment upstream of the passing lane L_u is determined by the actual or planned placement of the passing lane within the analysis segment. The length of the downstream highway segment within the effective length of the passing lane L_{de} is determined from Exhibit 15-23. Any remaining length of the analysis segment downstream of the passing lane is included in L_d, which is computed from Equation 15-19:

Equation 15-19

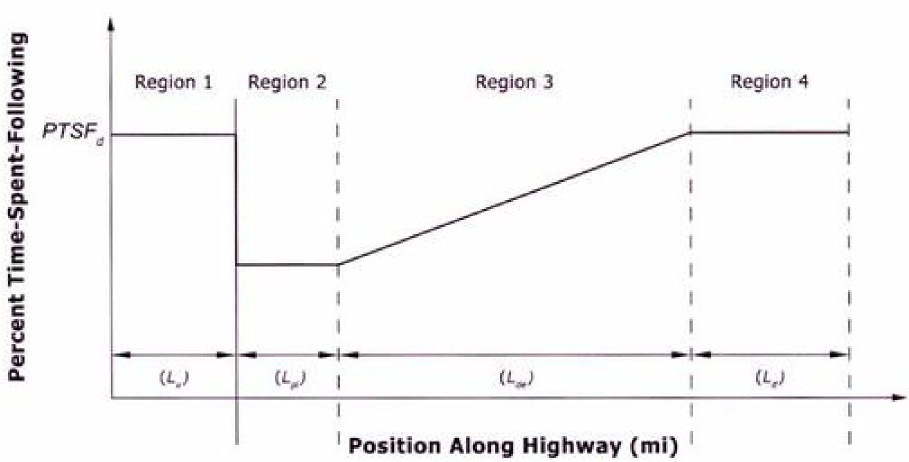
$$L_d = L_t - \left(L_u + L_{pl} + L_{de}\right)$$

where L_t is the total length of the analysis segment in miles and all other terms are as previously defined.

Step 3: Determine the PTSF

PTSF within lengths L_u and L_d is assumed to be equal to the $PTSF_d$ as predicted by the core analysis procedure (without a passing lane). Within the segment with the passing lane L_{pl}, PTSF is generally equal to 58% to 62% of its upstream value. This effect is a function of the directional demand flow rate. Within L_{de}, the PTSF is assumed to increase linearly from the passing lane value to the normal upstream value. This distribution is illustrated in Exhibit 15-25.

Exhibit 15-25
Effect of a Passing Lane on PTSF



On the basis of this model, the PTSF for the entire analysis segment, as affected by the passing lane, is given by Equation 15-20:

Equation 15-20

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + L_d + f_{pl,PTSF} L_{pl} + \left(\frac{1 + f_{pl,PTSF}}{2} \right) L_{de} \right]}{L_t}$$

where

$PTSF_{pl}$ = percent time-spent-following for segment as affected by the presence of a passing lane (decimal); and

$f_{pl,PTSF}$ = adjustment factor for the impact of a passing lane on percent time-spent-following, from Exhibit 15-26.

All other variables are as previously defined.

Exhibit 15-26
Adjustment Factor for the Impact of a Passing Lane on PTSF ($f_{pl,PTSF}$)

Directional Demand Flow Rate, v_d (pc/h)	$f_{pl,PTSF}$
≤100	0.58
200	0.59
300	0.60
400	0.61
500	0.61
600	0.61
700	0.62
800	0.62
≥900	0.62

Note: Interpolation is not recommended; use closest value.

If the analysis segment cannot encompass the entire length L_{de} because it is truncated by a town or major intersection within it, distance L_d is not used. Therefore, the actual downstream length within the analysis segment L'_{de} is less than the value of L_{de} tabulated in Exhibit 15-23. In this case, Equation 15-21 should be used instead of Equation 15-20:

Equation 15-21

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + f_{pl,PTSF} L_{pl} + f_{pl,PTSF} L'_{de} + \left(\frac{1 + f_{pl,PTSF}}{2} \right) \left(\frac{L'^{2}_{de}}{L_{de}} \right) \right]}{L_t}$$

where L_t is the total length of the analysis segment in miles and all other terms are as previously defined.

In general, the effective downstream distance of the passing lane should not be truncated. A downstream boundary short of the effective downstream distance should be considered at the point where any of the following occur:

- The environment of the highway radically changes, as in the case of entering a small town or developed area from a rural segment;
- A major unsignalized intersection is present, leading to a change in the demand flow rate;
- A proximate signalized intersection begins to affect the operation of the two-lane segment;
- The terrain changes significantly; or
- Lane or shoulder widths change significantly.

Step 4: Determine the ATS

The ATS within lengths L_u and L_d is assumed to be equal to ATS_d, the speed that would exist without the passing lane. Within the passing lane, the ATS is generally between 8% and 11% higher than its upstream value, depending on the directional demand flow rate. Within the effective downstream length, L_{de}, ATS is assumed to decrease linearly with the distance from the passing lane, from the passing lane value to the normal value. Exhibit 15-27 illustrates the impact of a passing lane on ATS.

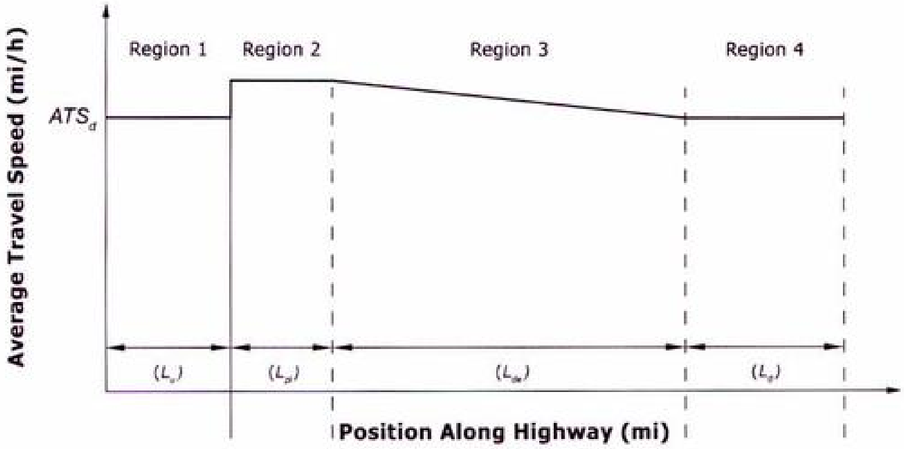


Exhibit 15-27
Impact of a Passing Lane on ATS

The ATS is computed with Equation 15-22:

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + L_d + \left(\frac{L_{pl}}{f_{pl,ATS}}\right) + \left(\frac{2L_{de}}{1 + f_{pl,ATS}}\right)}$$

Equation 15-22

where

ATS_{pl} = average travel speed in the analysis segment as affected by a passing lane (mi/h); and

$f_{pl,ATS}$ = adjustment factor for the effect of a passing lane on ATS, from Exhibit 15-28.

All other variables are as previously defined.

Exhibit 15-28
Adjustment Factor for Estimating the Impact of a Passing Lane on ATS ($f_{pl,ATS}$)

Directional Demand Flow Rate, v_d(pc/h)	$f_{pl,ATS}$
≤100	1.08
200	1.09
300	1.10
400	1.10
500	1.10
600	1.11
700	1.11
800	1.11
≥900	1.11

Note: Interpolation is not recommended; use closest value.

In the case where the analysis segment cannot include all of the effective downstream distance, L_{de}, because a town or major intersections cause the segment to be truncated, distance L'_{de} is less than the value of L_{de}. In this case, Equation 15-23 is used instead of Equation 15-22 to compute ATS.

Equation 15-23

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + \frac{L_{pl}}{f_{pl,ATS}} + \frac{2L'_{de}}{\left[1 + f_{pl,ATS} + \left(f_{pl,ATS} - 1\right)\left(\frac{L_{de} - L'_{de}}{L_{de}}\right)\right]}}$$

where all terms are as previously defined.

Step 5: Determine the LOS

Determining the LOS for a segment with a passing lane is no different from determining the LOS for a normal segment, except that ATS_{pl} and $PTSF_{pl}$ are used as the service measures with the criteria of Exhibit 15-3.

As with a normal segment, LOS for Class I highways is based on both PTSF and ATS. LOS for Class II highways is based only on PTSF. Class III highways would not normally have passing lanes, but if such a situation arose, PFFS = ATS/FFS would be used to determine LOS.

CLIMBING LANES

A *climbing lane* is a lane added on an upgrade on a two-lane highway to allow traffic to pass heavy vehicles whose speeds are reduced. Generally, the lane is added to the right, and all slow-moving vehicles should move to this lane, allowing faster vehicles to pass in the normal lane.

The American Association of State Highway and Transportation Officials (*8*) indicates that climbing lanes on two-lane highways are warranted when

High crash frequencies may also justify a climbing lane.

- The directional flow rate on the upgrade exceeds 200 veh/h;
- The directional flow rate for trucks on the upgrade exceeds 20 veh/h; and
- Any of the following conditions apply:
    - A speed reduction of 10 mi/h or more exists for a typical truck;
    - LOS E or F exists on the upgrade without a climbing lane; or
    - Without a climbing lane, the LOS is two or more levels lower on the upgrade than on the approach segment to the grade.

An operational analysis of the impact of a climbing lane on a two-lane highway is performed by using the procedure for passing lanes given above, with three major differences:

1. Adjustment factors for the existence of the climbing lane are taken from Exhibit 15-29,
2. The analysis without a climbing lane is conducted by using the specific grade procedures, and
3. Distances L_u and L_d are set to zero.

The effective downstream distance L_{dc} is also generally set to zero unless the climbing lane ends before the grade does. In this case, a value less than the values given previously in Exhibit 15-23 should be considered.

Exhibit 15-29
Adjustment Factors (f_{pl}) for Estimating ATS and PTSF Within a Climbing Lane

Directional Demand Flow Rate, v_d (pc/h)	f_{pl} ATS	f_{pl} PTSF
0–300	1.02	0.20
>300–600	1.07	0.21
>600	1.14	0.23

DESIGN AND OPERATIONAL TREATMENTS

Two-lane highways make up approximately 80% of all paved rural highways in the United States but carry only about 30% of all traffic. For the most part, two-lane highways carry light volumes and experience few operational problems. However, some two-lane highways periodically experience significant operational and safety problems brought about by a variety of traffic, geometric, and environmental causes. Such highways may require design or operational improvements to alleviate congestion.

When traffic operational problems occur on two-lane highways, many agencies consider widening to four lanes. Another effective method for alleviating operational problems is to provide passing lanes at intervals in each direction of travel or to provide climbing lanes on steep upgrades. Passing and climbing lanes cannot increase the capacity of a two-lane highway, but they can improve its LOS. Short sections of four-lane highway can function as a pair of passing lanes in opposite directions of travel. Operational analysis procedures for passing and climbing lanes were provided previously in this section.

A number of other design and operational treatments are effective in alleviating operational congestion on two-lane highways, including

- Turnouts,
- Shoulder use,
- Wide cross sections,
- Intersection turn lanes, and
- Two-way left-turn lanes (TWLTLs).

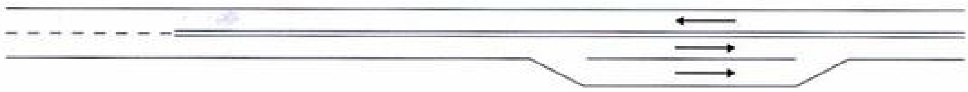
No calculation methodologies are provided in this section for these treatments; however, the treatments are discussed below to indicate their potential for improving traffic operations on two-lane highways.

Turnouts

A turnout is a widened, unobstructed shoulder area on a two-lane highway that allows slow-moving vehicles to pull out of the through lane so that vehicles following may pass. Turnouts are relatively short, generally less than 625 ft. At a turnout, the driver of a slow-moving vehicle that is delaying one or more following vehicles is expected to pull out of the through lane, allowing the vehicles to pass. The driver of the slow-moving vehicle is expected to remain in the turnout only long enough to allow the following vehicles to pass before returning to the travel lane. When there are only one or two following vehicles, this maneuver can usually be completed smoothly, with no need for the vehicle to stop in the turnout. When there are three or more following vehicles, the vehicle in the turnout will generally have to stop to allow all vehicles to pass. In this case, the driver of the slower vehicle is expected to stop before the end of the turnout, so that the vehicle will develop some speed before reentering the lane. Signs inform drivers of the turnout's location and reinforce the legal requirements concerning turnout use.

Turnouts have been used in several countries to provide additional passing opportunities on two-lane highways. In the United States, turnouts have been used extensively in western states. Exhibit 15-30 illustrates a typical turnout.

Exhibit 15-30
Typical Turnout Illustrated



Turnouts may be used on nearly any type of two-lane highway that offers limited passing opportunities. To avoid confusing drivers, turnouts and passing lanes should not be intermixed on the same highway.

A single well-designed and well-located turnout can be expected to accommodate 20% to 50% of the number of passes that would occur in a 1.0-mi passing lane in level terrain (*7, 9*). Turnouts have been found to operate safely, with experts (*9–11*) noting that turnout accidents occur at a rate of only 1 per 80,000 to 400,000 users.

Shoulder Use

The primary purpose of the shoulder on two-lane highways is to provide a stopping and recovery area for disabled or errant vehicles. However, paved shoulders also may be used to increase passing opportunities on two-lane highways.

In some parts of the United States and Canada, if the paved shoulders are adequate, there is a long-standing custom for slower vehicles to move to the shoulder when a vehicle approaches from the rear. The slower vehicle returns to the travel lane once the passing vehicle has cleared. The custom is regarded as a courtesy and requires little or no sacrifice in speed by either motorist. A few highway agencies encourage drivers of slow-moving vehicles to use the shoulder in this way because it improves the LOS of two-lane highways without the expense of adding passing lanes or widening the highway. On the other hand, there are agencies that discourage this practice because their shoulders are not designed for frequent use by heavy vehicles.

One highway agency in the western United States generally does not permit shoulder use by slow-moving vehicles but designates specific sections on which the shoulder may be used for this purpose. These shoulder segments range in length from 0.2 to 3.0 mi and are identified by traffic signs.

Research (*7, 9*) has shown that a shoulder-use segment is about 20% as effective in reducing platoons as a passing lane of comparable length.

Wide Cross Sections

Two-lane highways with lanes about 50% wider than normal have been used in several European countries as a less expensive alternative to passing lanes. Sweden, for example, built approximately 500 mi of roadways with two 18-ft travel lanes and relatively narrow (3.3-ft) shoulders. The wider lane permits faster vehicles to pass slower vehicles while encroaching only slightly on the opposing lane of traffic. Opposing vehicles must move toward the shoulder to permit such maneuvers. Roadway segments with wider lanes can be provided at intervals, like passing lanes, to increase passing opportunities on two-lane highways.

Research has shown that speeds at low traffic volumes tend to increase on wider lanes, but the effect on speeds at higher volumes varies (*12*). More than 70% of drivers indicated that they appreciate the increased passing opportunities available on the wider lanes. No safety problems have been associated with the wider lanes.

Formal procedures have not yet been developed for evaluating the traffic operational effectiveness of wider lanes in increasing the passing opportunities on a two-lane highway. The traffic operational performance on a directional two-lane highway segment containing wider lanes can reasonably be estimated as midway between the segment with and without a passing lane of comparable length.

Intersection Turn Lanes

Intersection turn lanes are desirable at selected locations on two-lane highways to reduce delays to through vehicles caused by turning vehicles and to reduce turning accidents. Separate right- and left-turn lanes may be provided, as appropriate, to remove turning vehicles from the through travel lanes. Left-turn lanes, in particular, provide a protected location for turning vehicles to wait for an acceptable gap in the opposing traffic stream. This reduces the potential for collisions from the rear and may encourage drivers of left-turning vehicles to wait for an adequate gap in opposing traffic before turning. Exhibit 15-31 shows a typical two-lane highway with left-turn lanes at an intersection.

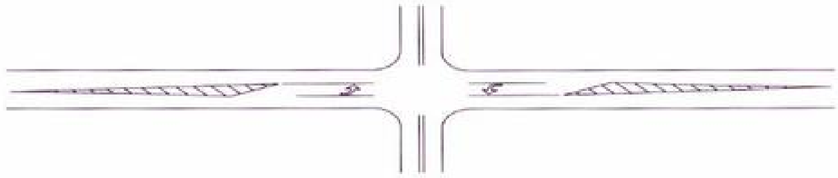
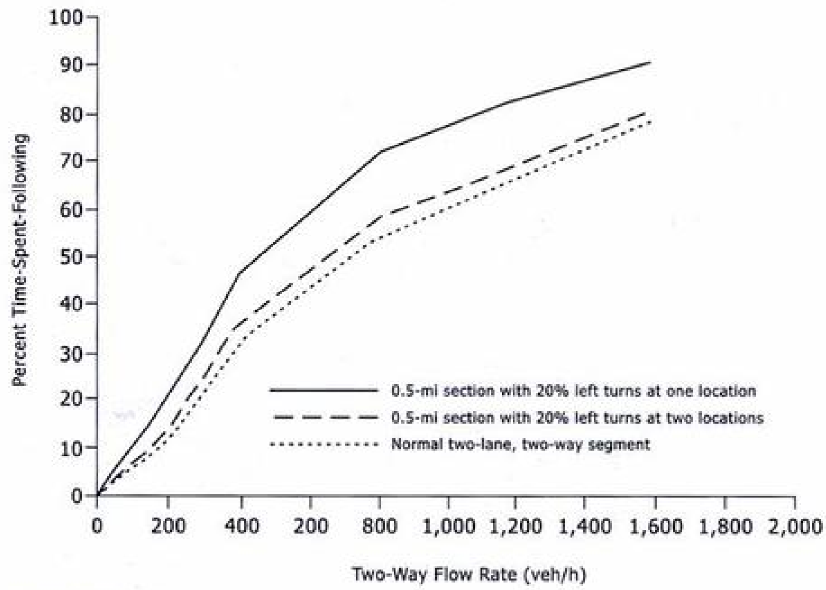


Exhibit 15-31
Typical Two-Lane Highway Intersection with Left-Turn Lane

Research recommends specific operational warrants for left-turn lanes at intersections on two-lane highways based on the directional volumes and the percentage of left turns (*13*). The HCM's intersection analysis methodologies can be used to quantify the effects of intersection turn lanes on signalized and unsignalized intersections. However, there is no methodology for estimating the effect of turn lanes on average highway speed. Modeling of intersection delays shows the relative magnitude of likely effects of turning delays on PTSF (*14*); the results are shown in Exhibit 15-32. The top line in the exhibit shows that turning vehicles can increase PTSF substantially over a short road segment. However, when these effects are averaged over a longer road segment, the increase in PTSF is greatly reduced, as indicated by the dashed line in the exhibit. The provision of intersection turn lanes could minimize these effects.

Exhibit 15-32
Illustrative Effect of Turning Delays at Intersections on PTSF

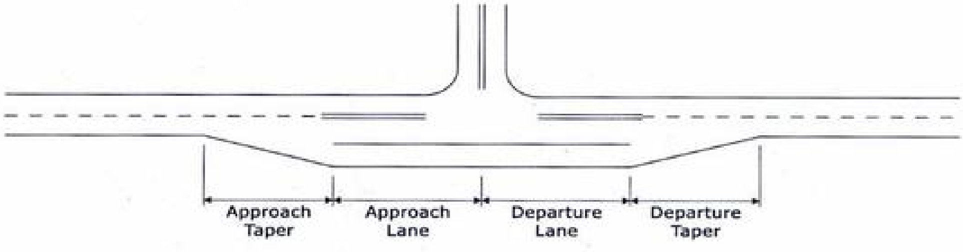


Source: Hoban (*14*).

Several agencies in the United States provide shoulder bypass lanes at three-leg intersections as a low-cost alternative to a left-turn lane. As shown in Exhibit 15-33, a portion of the paved shoulder may be marked as a lane for through traffic to bypass vehicles that are slowing or stopped to make a left turn. Bypass lanes may be appropriate for intersections that do not have volumes high enough to warrant a left-turn lane.

The delay benefits of shoulder bypass lanes have not been quantified, but field studies have indicated that 97% of drivers who need to avoid delay will make use of an available shoulder bypass lane. One state has reported a marked decrease in rear-end collisions at intersections where shoulder bypass lanes were provided (*15*).

Exhibit 15-33
Typical Shoulder Bypass Lane at a Three-Leg Intersection on a Two-Lane Highway

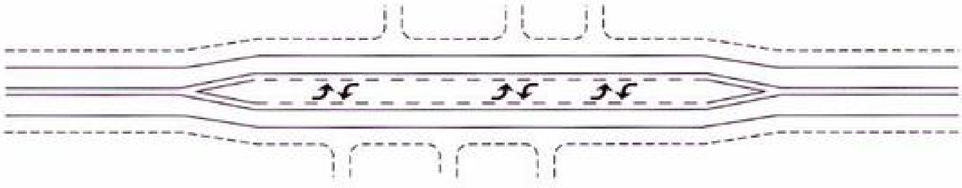


Two-Way Left-Turn Lanes

A TWLTL is a paved area in the highway median that extends continuously along a roadway segment and is marked to provide a deceleration and storage area for vehicles traveling in either direction that are making left turns at intersections and driveways.

TWLTLs have been used for many years on urban and suburban streets with high driveway densities and turning demands to improve safety and reduce delays to through vehicles. TWLTLs can be used on two-lane highways in rural and urban fringe areas to obtain the same types of operational and safety benefits—particularly on Class III two-lane highways. Exhibit 15-34 illustrates a typical TWLTL.

Exhibit 15-34
Typical TWLTL on a Two-Lane Highway

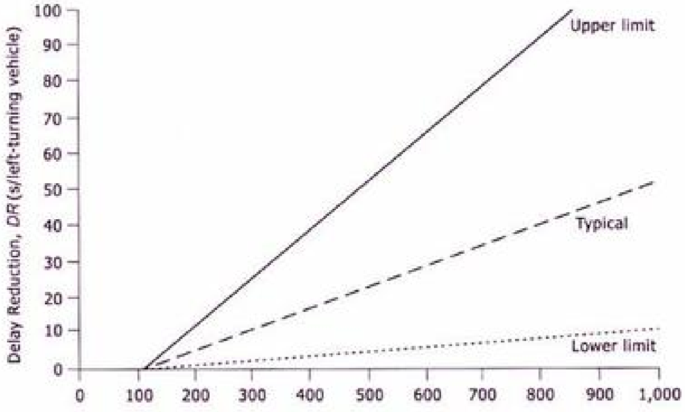


There is no formal methodology for evaluating the traffic operational effectiveness of a TWLTL on a two-lane highway. Research has found that delay reduction provided by a TWLTL depends on both the left-turn demand and the opposing traffic volume (*9*). Without a TWLTL or other left-turn treatment, vehicles that are slowing or stopped to make a left turn may create delays for following through vehicles. A TWLTL minimizes these delays and makes the roadway segment operate more like two-way and directional segments with 100% no-passing zones. These research results apply to sites that do not have paved shoulders available for following vehicles to bypass turning vehicles. Paved shoulders may alleviate as much of the delay as a TWLTL.

Research has found little delay reduction at rural TWLTL segments with traffic volumes below 300 veh/h in one direction (*9*). At several low-volume sites, no reduction was observed. The highest delay reduction observed was 3.4 s per left-turning vehicle. Therefore, at low-volume rural sites, TWLTLs should be considered for reducing accidents but should not be expected to improve the operational performance of the highway.

At higher-volume urban fringe sites, greater delay reduction was found with TWLTLs on a two-lane highway. Exhibit 15-35 shows the expected delay reduction per left-turning vehicle as a function of opposing volume. As the delay reduction increases, a TWLTL can be justified for improving both safety and operations.

Exhibit 15-35
Estimated Delay Reduction with a TWLTL on a Two-Lane Highway Without Paved Shoulders



Source: Harwood and St. John (*9*).

5. BICYCLE METHODOLOGY

The calculation of bicycle LOS on multilane and two-lane highways shares the same methodology, since multilane and two-lane highways operate in fundamentally the same manner for bicyclists. Cyclists travel much more slowly than the prevailing traffic flow, staying as far to the right as possible and using paved shoulders when available, which indicates the need for only one model.

The bicycle LOS model for two-lane and multilane highways uses a traveler perception model calibrated by using a linear regression (*16*). The model fits independent variables associated with roadway characteristics to the results of a field-based user survey that rated the comfort of various bicycle facilities. The resulting bicycle LOS score generally ranges from 0.5 to 6.5 and is stratified to produce a LOS A–F result, on the basis of Exhibit 15-4.

SCOPE OF THE METHODOLOGY

Spatial and Temporal Limits

The bicycle method applies to paved shoulders, bicycle lanes, and shared lanes on two-lane highways. These facility types were illustrated in Exhibit 3-24 in Chapter 3, Modal Characteristics. Sidepaths are not addressed by the method, but they could be treated as an off-street facility, if located sufficiently far away from the highway, as described in Section 1 of Chapter 24, Off-Street Pedestrian and Bicycle Facilities.

Segment boundaries should be established at points where a change occurs in any of the following: terrain, lane widths, shoulder width, facility classification, or demand flow rate. If both a bicycle and a motorized vehicle analysis are being performed for the two-lane highway, the segments used for the two analyses should be identical. The recommended length of the analysis period is the HCM standard of 15 min (although longer periods can be examined).

Performance Measures

This method produces the following performance measures:

- Bicycle LOS score, and
- LOS based on the bicycle LOS score.

Strengths of the Methodology

The bicycle LOS score and letter produced by this method are sensitive to bicyclist separation from motor vehicle traffic, motorized traffic volumes and speed, heavy vehicle presence, and pavement condition. The bicycle LOS score and letter can be directly compared with the modal LOS scores and letters produced by other HCM traveler perception–based methods, such as those found in many of the urban street and intersection chapters in Volume 3.

Limitations of the Methodology

This methodology was developed with data collected on urban and suburban streets, including facilities that would be defined as suburban two-lane highways. Although the methodology has been successfully applied to rural two-lane highways in different parts of the United States, users should be aware that conditions on many rural two-lane highways will be outside the range of values used to develop the bicycle LOS model. The ranges of values used in the development of the bicycle LOS model (*16*) are shown below:

- Width of the outside through lane: 10 to 16 ft;
- Shoulder width: 0 to 6 ft;
- Motorized vehicle volumes: up to 36,000 annual average daily traffic (AADT);
- Posted speed: 25 to 50 mi/h;
- Heavy vehicle percentage: 0% to 2%; and
- Pavement condition: 2 to 5 on the Federal Highway Administration (FHWA) 5-point pavement rating scale (*17*).

The bicycle LOS methodology does not take differences in prevalent driver behavior into consideration, although driver behavior may vary considerably both regionally and by facility. In particular, the likelihood of drivers slowing down or providing additional horizontal clearance while passing cyclists plays a significant role in the perceived quality of service of a facility.

REQUIRED DATA AND SOURCES

Exhibit 15-36 lists the information necessary to apply the bicycle methodology and suggests potential sources for obtaining these data. As can be seen in the exhibit, many of the input data required for a bicycle analysis are also required for a motorized vehicle analysis.

Exhibit 15-36 also suggests default values for use when segment-specific information is not available. The user is cautioned that every use of a default value instead of a field-measured, segment-specific value may make the analysis results more approximate and less related to the specific conditions that describe the highway. HCM defaults should only be used when (*a*) field data cannot be collected and (*b*) locally derived defaults do not exist.

Exhibit 15-36
Required Input Data, Potential Data Sources, and Default Values for Two-Lane and Multilane Highway Bicycle Analysis

Required Data	Potential Data Source(s)	Suggested Default Value
	Geometric Data	
Lane width (ft)*	Road inventory, aerial photo	12 ft
Shoulder width (ft)*	Road inventory, aerial photo	6 ft
Speed limit (mi/h)	Field data, road inventory	Must be provided
Number of directional through lanes	Field data, road inventory	1 (two-lane highways) 2 (multilane highways)
Pavement condition[a] (FHWA 5-point scale)	Field data, pavement condition inventory	4 (good)
	Demand Data	
Hourly motor vehicle demand (veh/h)*	Field data, past counts, models	Must be provided
Directional volume split (%)*	Field data, past counts, models	Must be provided
Analysis period length (min)*	Set by analyst	15 min (0.25 h)
Peak hour factor (decimal)*	Field data	0.88
Heavy vehicle percentage (decimal)*	Field data, past counts	0.06[b]
Percent of segment with occupied on-highway parking (decimal)[c]	Field data	0.00

Notes: ***Bold italic*** indicates high sensitivity (±2 LOS letters) of LOS to the choice of default value.
Bold indicates moderate sensitivity (±1 LOS letter) of LOS to the choice of default value.
*Also used by the two-lane highway motorized vehicle method.
[a] Sensitivity reflects pavement conditions 2–5. Very poor pavement (i.e., 1) typically results in LOS F, regardless of other input values.
[b] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.
[c] Moderate sensitivity on Class III two-lane highways.

OVERVIEW OF THE METHODOLOGY

Exhibit 15-37 illustrates the steps involved in the bicycle methodology.

Exhibit 15-37
Flowchart of the Bicycle Methodology for Two-Lane and Multilane Highways

Step 1: Gather Input Data
Lane and shoulder width
Number of directional lanes
Speed limit
Pavement condition
Automobile demand data
On-highway parking occupancy

↓

Step 2: Calculate the Directional Flow Rate in the Outside Lane
(Equation 15-24)
Automobile demand, number of directional lanes, peak hour factor

↓

Step 3: Calculate the Effective Width
(Equations 15-25 to 15-29)
Lane and shoulder width
Automobile demand volume
On-highway parking occupancy

↓

Step 4: Calculate the Effective Speed Factor
(Equation 15-30)
Speed limit

↓

Step 5: Determine the LOS
(Equation 15-31 and Exhibit 15-4)
Directional flow rate in the outside lane
Effective speed factor
Heavy vehicle percentage
Pavement condition
Effective width

COMPUTATIONAL STEPS

Step 1: Gather Input Data

Exhibit 15-36 listed the information that must be available before a two-lane or multilane highway segment can be analyzed and potential sources for that information. The exhibit also suggested default values for use when neither site-specific data nor local default values are available.

Pavement rating is determined by using FHWA's 5-point present serviceability rating scale (*17*): 1 (very poor), 2 (poor), 3 (fair), 4 (good), and 5 (very good).

Step 2: Calculate the Directional Flow Rate in the Outside Lane

On the basis of the hourly directional volume, the peak hour factor, and the number of directional lanes (one for basic two-lane highways, two or more for passing lanes or multilane highways), calculate the directional demand flow rate of motorized traffic in the outside lane with Equation 15-24:

$$v_{OL} = \frac{V}{PHF \times N}$$

Equation 15-24

where

v_{OL} = directional demand flow rate in the outside lane (veh/h),

V = hourly directional volume (veh/h),

PHF = peak hour factor, and

N = number of directional lanes (=1 for two-lane highways).

Step 3: Calculate the Effective Width

The effective width of the outside through lane depends on both the actual width of the outside through lane and the shoulder width, since cyclists will be able to travel in the shoulder where one is provided. Moreover, striped shoulders of 4 ft or greater provide more security to cyclists by giving cyclists a dedicated place to ride outside of the motorized vehicle travelway. Thus, an 11-ft lane and adjacent 5-ft paved shoulder results in a larger effective width for cyclists than a 16-ft lane with no adjacent shoulder.

Parking occasionally exists along two-lane highways, particularly in developed areas (Class III highways) and near entrances to recreational areas (Class II and Class III highways) where a fee is charged for off-highway parking or where the off-highway parking is inadequate for the parking demand. On-highway parking reduces the effective width, because parked vehicles take up shoulder space and bicyclists leave some shy distance between themselves and the parked cars.

Equation 15-25 through Equation 15-29 are used to calculate the effective width W_e on the basis of the paved shoulder width W_s and the hourly directional volume V:

If W_s is greater than or equal to 8 ft:

Equation 15-25

$$W_e = W_v + W_s - (\%OHP \times 10\text{ ft})$$

If W_s is greater than or equal to 4 ft and less than 8 ft:

Equation 15-26

$$W_e = W_v + W_s - 2 \times [\%OHP(2\text{ ft} + W_s)]$$

If W_s is less than 4 ft:

Equation 15-27

$$W_e = W_v + [\%OHP(2\text{ ft} + W_s)]$$

with, if V is greater than 160 veh/h:

Equation 15-28

$$W_v = W_{OL} + W_s$$

Otherwise,

Equation 15-29

$$W_v = (W_{OL} + W_s) \times (2 - 0.005V)$$

where

W_v = effective width as a function of traffic volume (ft),

W_{OL} = outside lane width (ft),

W_s = paved shoulder width (ft),

V = hourly directional volume per lane (veh/h),

W_e = average effective width of the outside through lane (ft), and

$\%OHP$ = percentage of segment with occupied on-highway parking (decimal).

Step 4: Calculate the Effective Speed Factor

The effect of motor vehicle speed on bicycle quality of service is primarily related to the differential between motor vehicle and bicycle travel speeds. For example, a typical cyclist may travel in the range of 15 mi/h. An increase in motor vehicle speeds from 20 to 25 mi/h is more readily perceived than a speed increase from 60 to 65 mi/h, since the speed differential increases by 100% in the first instance compared with only 11% in the latter. Equation 15-30 shows the calculation of the effective speed factor that accounts for this diminishing effect.

Equation 15-30

$$S_t = 1.1199 \ln(S_p - 20) + 0.8103$$

where

S_t = effective speed factor, and

S_p = posted speed limit (mi/h).

Step 5: Determine the LOS

With the results of Steps 1–4, the bicycle LOS score can be calculated from Equation 15-31:

Equation 15-31

$$BLOS = 0.507 \ln(v_{OL}) + 0.1999S_t(1 + 10.38HV)^2 + 7.066(1/P)^2 - 0.005(W_e)^2 + 0.760$$

where

$BLOS$ = bicycle level of service score;

v_{OL} = directional demand flow rate in the outside lane (veh/h);

S_t = effective speed factor;

HV = percentage of heavy vehicles (decimal); if $V < 200$ veh/h, then HV should be limited to a maximum of 50%;

P = FHWA's 5-point pavement surface condition rating; and

W_e = average effective width of the outside through lane (ft).

Finally, the BLOS score value is used in Exhibit 15-4 to determine the bicycle LOS for the segment.

6. APPLICATIONS

EXAMPLE PROBLEMS

Section 7 of Chapter 26, Freeway and Highway Segments: Supplemental, provides five example problems that go through each of the computational steps involved in applying the motorized vehicle and bicycle methods:

1. Class I highway LOS,
2. Class II highway LOS,
3. Class III highway LOS,
4. Class I highway LOS with a passing lane, and
5. Two-lane highway bicycle LOS.

RELATED CONTENT IN THE HCMAG

The *Highway Capacity Manual Applications Guide* (HCMAG), accessible through the online HCM Volume 4, provides guidance on applying the HCM on two-lane highways. Case Study 3 goes through the process of identifying the goals, objectives, and analysis tools for investigating LOS on Krome Avenue, a 33-mile route that bypasses Miami, Florida, on its west side. The case study applies the analysis tools to assess the performance of the route, to identify areas that are deficient, and to investigate alternatives for correcting the deficiencies.

This case study includes the following problems related to two-lane highways:

1. Determination of facility types for analysis
   a. At what point does Krome Avenue change from a two-lane highway to a signalized arterial?
   b. What highway class should be assigned to each of the segments?
   c. What, if any, conditions exist at the controlled intersections that could affect the analyses?
2. Planning methodology or service volume table application
   a. North section: Class I two-lane highway
   b. Center section: Class I or II two-lane highway
3. Application of HCM chapters to arterial and highway segments
   a. North section: Class I two-lane highway
   b. Center section: Class I or II two-lane highway

Other problems in the case study evaluate options for a problematic intersection along the highway and evaluate the south section of the highway, which is treated as an urban street because of its 1-mile traffic signal spacing.

Although the HCMAG was based on the HCM2000's procedures and chapter organization, the general thought process described in its case studies is also applicable to this edition of the HCM.

EXAMPLE RESULTS

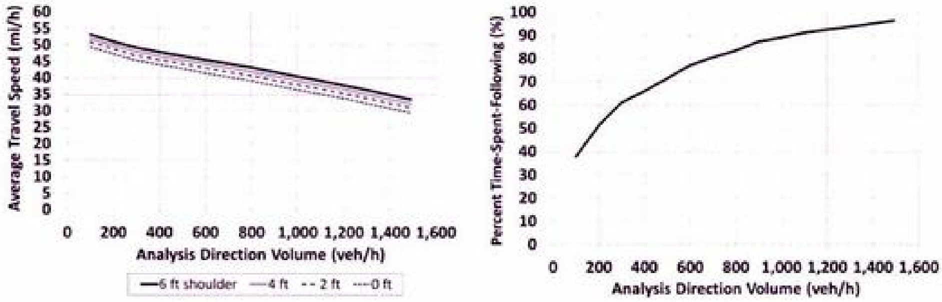
This section presents the results of applying this chapter's method in typical situations. Analysts can use the illustrative results presented in this section to observe the sensitivity of output performance measures to various inputs, as well as to help evaluate whether their analysis results are reasonable. The exhibits in this section are not intended to substitute for an actual analysis and are deliberately provided in a format large enough to depict general trends in the results, but not large enough to pull out specific results.

Motorized Vehicle Mode

Sensitivity of Results to Shoulder Width

Exhibit 15-38 presents illustrative results showing how ATS and PTSF vary with increasing analysis direction volume and decreasing shoulder width. ATS is relatively insensitive to shoulder width, and PTSF is not affected at all by shoulder width. As directional volumes increase, ATS decreases, while PTSF increases.

Exhibit 15-38
Illustrative Effect of Volume and Shoulder Width on ATS and PTSF



(a) ATS (b) PTSF

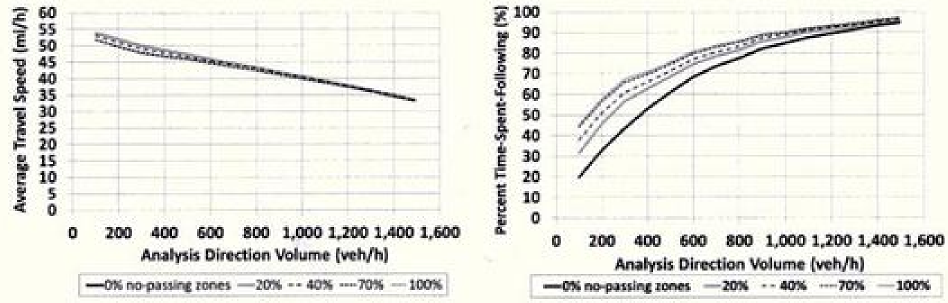
Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane widths, 8 access points per mile, 60-mi/h base free-flow speed, level terrain, 40% no-passing zones, 60/40 directional split, 6% trucks, 0.88 PHF, no passing lanes, and 10-mi segment length. PTSF is not sensitive to shoulder width.

Sensitivity of Results to Percent No-Passing Zones

Exhibit 15-39 presents illustrative results showing how ATS and PTSF vary with increasing analysis direction volume and increasing percentage of no-passing zones. ATS is not very sensitive to percent no-passing zones at low volumes and, for all practical purposes, is insensitive at moderate and high directional volumes. Increasing the percentage of no-passing zones has no effect on the ATS result until the value increases above 20%.

PTSF is particularly sensitive to percentage of no-passing zones at low and moderate volumes, but its sensitivity declines as directional volumes increase. In addition, the PTSF results for the highest percentage of no-passing-zone values used in the exhibit (70% and 100%) are similar throughout the range of directional volumes, indicating that a substantial number of passing opportunities need to be provided to have an impact on PTSF.

Exhibit 15-39
Illustrative Effect of Volume and Percent No-Passing Zones on ATS and PTSF



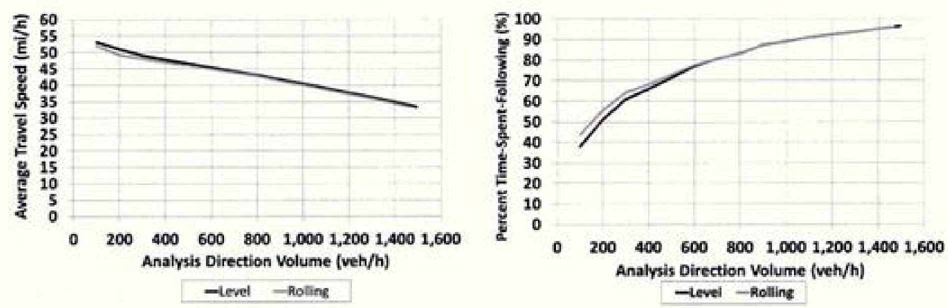
(a) ATS (b) PTSF

Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane widths, 6-foot shoulders, 8 access points per mile, 60-mi/h base free-flow speed, level terrain, 60/40 directional split, 6% trucks, 0.88 PHF, no passing lane, and 10-mi segment length.
ATS values for 0% and 20% no-passing zones are identical.

Sensitivity of Results to General Terrain Type

Exhibit 15-40 shows that the general terrain type has little impact on ATS and PTSF. In addition, the directional capacity of the roadway is slightly lower in rolling terrain than in level terrain. Note that the percentage of no-passing zones is typically related to terrain and, on the basis of Exhibit 15-39(b), has a more substantial effect on PTSF than does the general terrain type.

Exhibit 15-40
Illustrative Effect of Volume and General Terrain Type on ATS and PTSF



(a) ATS (b) PTSF

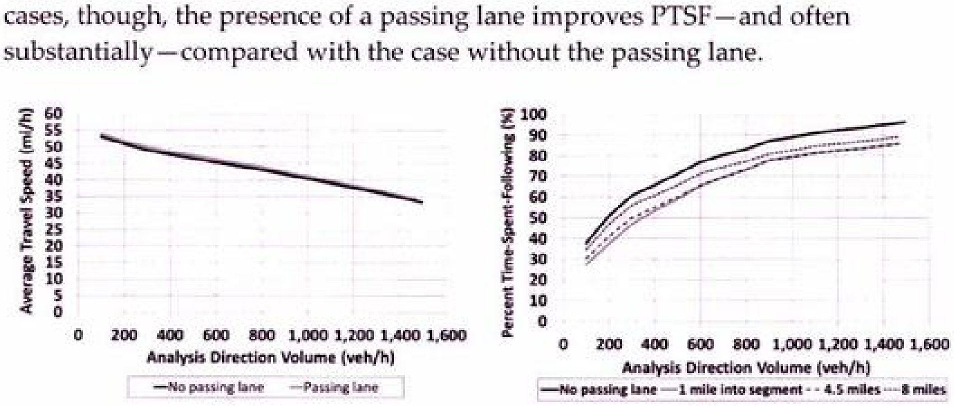
Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane widths, 6-foot shoulders, 8 access points per mile, 40% no-passing zones, 60-mi/h base free-flow speed, 60/40 directional split, 6% trucks, 0.88 PHF, no passing lane, and 10-mi segment length.

Sensitivity of Results to Passing Lane Location

Exhibit 15-41 illustrates the impact on ATS and PTSF of the location of a 1-mi passing lane along a 10-mi segment of two-lane highway (e.g., a length of highway between two towns located 10 mi apart). The farther the passing lane is located into the segment, the more likely that not all of its effective length will be usable in this case, where the highway's character changes at the segment's end.

Exhibit 15-41(a) shows that the presence of a passing lane produces a tiny increase in ATS, regardless of the passing lane's location. Exhibit 15-41(b) shows that—at low directional volumes—the earlier in the segment the passing lane occurs, the more pronounced the effect on PTSF. At high directional volumes, the passing lane location has less of an effect on PTSF, both because of the difficulty of passing before and after the passing lane and because the assumed passing lane length (1 mi) is less than the optimal length shown in Exhibit 15-24. In all cases, though, the presence of a passing lane improves PTSF—and often substantially—compared with the case without the passing lane.

Exhibit 15-41
Illustrative Effect of Volume and Passing Lane Location on ATS and PTSF



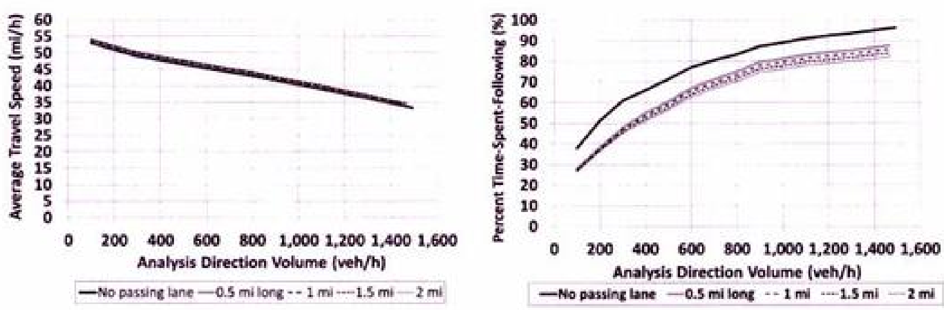
(a) ATS (b) PTSF

Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane widths, 6-foot shoulders, 8 access points per mile, 40% no-passing zones, 60-mi/h base free-flow speed, level terrain, 60/40 directional split, 6% trucks, 0.88 PHF, 1-mi passing lane length, and 10-mi segment length.

Sensitivity of Results to Passing Lane Length

Exhibit 15-42 illustrates the impact on ATS and PTSF of the length of a passing lane located 1 mi into a 10-mi segment of two-lane highway. Exhibit 15-42(a) shows that ATS increases slightly as the passing lane length increases. Exhibit 15-42(b) shows that any passing lane length lowers the PTSF, compared with the case without a passing lane, but that longer passing lane lengths result in greater reductions in PTSF as directional volumes increase.

Exhibit 15-42
Illustrative Effect of Volume and Passing Lane Length on ATS and PTSF



(a) ATS (b) PTSF

Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane widths, 6-foot shoulders, 8 access points per mile, 40% no-passing zones, 60-mi/h base free-flow speed, level terrain, 60/40 directional split, 6% trucks, 0.88 PHF, passing lane starting 1 mi into segment, and 10-mi segment length.

Other Observations

The selected highway class has no impact on the calculated values of ATS and PTSF. However, the class does make a difference in the calculation of LOS on the basis of PTSF or the combination of PTSF and ATS. The LOS thresholds for PTSF are 5 percentage points higher for Class II highways than for Class I highways.

No comparisons were presented above for PFFS, which is the service measure for Class III highways. However, because PFFS is simply ATS divided by the free-flow speed, all the comparisons related to ATS given above also apply to Class III highways.

Bicycle Mode

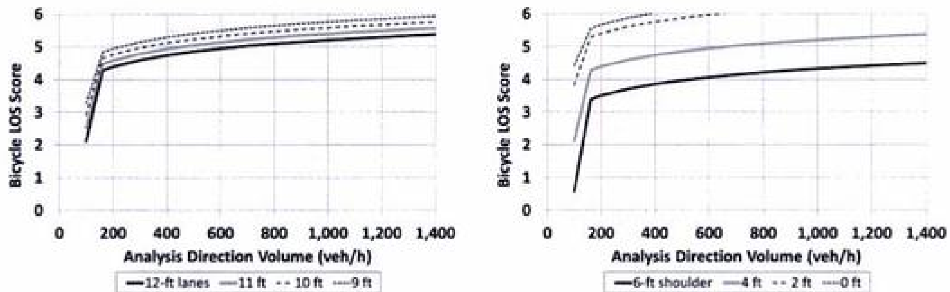
Sensitivity of Results to Lane and Shoulder Width

Exhibit 15-43 depicts how the BLOS score is affected by different lane and shoulder widths at different directional volumes. As was shown in Exhibit 15-4, lower BLOS score values indicate that bicyclists perceive better conditions, with the LOS A/B threshold set at a BLOS score of 1.5, the LOS E/F threshold set at 5.5, and each 1.0 increase in the BLOS score indicating a one-letter drop in the level of service.

Comparison of Exhibit 15-43(a) with Exhibit 15-43(b) indicates that the relative change in the BLOS score with either a reduced lane width or a reduced shoulder width is the same at any volume. In addition, shoulder width has a greater impact on the BLOS score than does the lane width.

Finally, the effect of low volumes on the BLOS score can be seen clearly in both graphs as a sharp drop in the score when the volume is 160 veh/h or less. This effect is a result of Equation 15-29, where the effective width as a function of traffic volume can be as much as twice the physical width.

Exhibit 15-43
Illustrative Effect of Volume, Lane Width, and Shoulder Width on BLOS Score



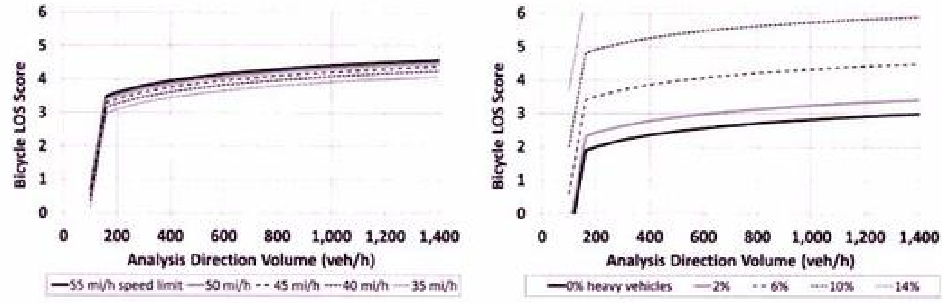
(a) Lane Width (b) Shoulder Width

Note: Calculated by using this chapter's methods. Fixed values include pavement condition = 4 (good), speed limit = 50 mi/h, PHF = 0.88, 6% heavy vehicles, and no on-highway parking. In Exhibit 15-43(a), shoulder width = 4 ft. In Exhibit 15-43(b), lane width = 12 ft.

Sensitivity of Results to Traffic Speed and Heavy Vehicles

Exhibit 15-44(a) shows that traffic speed has a relatively small effect on the BLOS score, within the range of speed limits typically found on two-lane highways. On the other hand, Exhibit 15-44(b) shows that the heavy vehicle percentage has a large effect on the BLOS score. When heavy vehicles form 14% of the traffic volume, LOS F (i.e., BLOS score > 5.5) conditions result at any traffic volume over 160 veh/h, for the conditions shown in the note accompanying the exhibit.

Exhibit 15-44
Illustrative Effect of Volume, Speed Limit, and Heavy Vehicles on BLOS Score



(a) Speed Limit (b) Heavy Vehicle Percentage

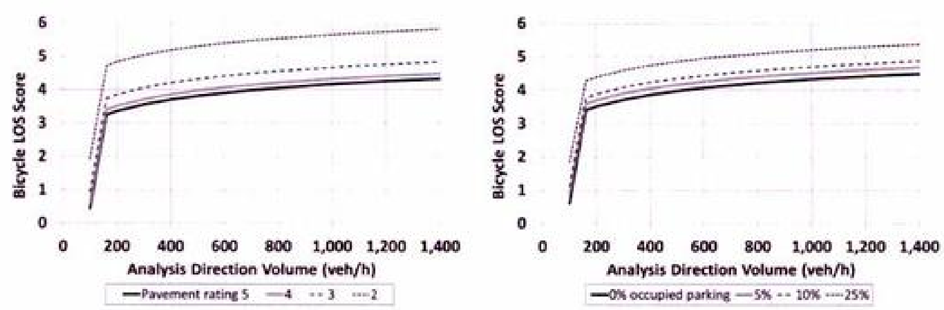
Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane width, 6-ft shoulder, pavement condition = 4 (good), PHF = 0.88, no on-highway parking. In Exhibit 15-43(a), the heavy vehicle percentage is 6%. In Exhibit 15-43(b), the speed limit is 50 mi/h.

Sensitivity of Results to Pavement Condition and On-Highway Parking

Exhibit 15-45(a) shows that the BLOS score is sensitive to the pavement condition rating. Furthermore, when the pavement rating is 1 (very poor), the BLOS score is off the graph, well into the LOS F range (e.g., BLOS = 6.51 with no traffic volume, for the conditions shown with the exhibit).

Exhibit 15-45(b) shows that, within the range of occupied parking that might be found along a Class II highway, on-highway parking has a small impact on the BLOS score. However, at the higher percentages of on-highway parking that might be found along a Class III highway, the impact of parking is greater.

Exhibit 15-45
Illustrative Effect of Volume, Pavement Condition, and On-Highway Parking on BLOS Score



(a) Pavement Condition (b) On-Highway Parking

Note: Calculated by using this chapter's methods. Fixed values include 12-ft lane width, 6-ft shoulder, 6% heavy vehicles, 50-mi/h speed limit, and PHF = 0.88. In Exhibit 15-45(a), there is no on-highway parking. In Exhibit 15-45(b), the pavement condition rating is 4 (good).
When the pavement condition is 1 (very poor), BLOS score > 5.5 (LOS F) across the full range of directional volumes, for the conditions listed above.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 15-46 shows generalized daily service volumes for use in planning and preliminary design. The exhibit provides daily service volume values for three types of segments: (*a*) a Class I highway in level terrain, (*b*) a Class I highway in rolling terrain, and (*c*) a Class II highway in rolling terrain.

Typical conditions assumed for each are given below the table. Various values of *K*- and *D*-factors are given. Since these values vary greatly from region to region, the analyst must select the values most appropriate to the particular application. Interpolation may be used, if desired, to obtain intermediate values.

Exhibit 15-46
Generalized Daily Service Volumes for Two-Lane Highways

K-Factor	D-Factor	Class I—Level				Class I—Rolling				Class II—Rolling			
		LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E	LOS B	LOS C	LOS D	LOS E
0.09	50%	5.5	9.3	16.5	31.2	4.2	8.4	15.7	30.3	5.0	9.8	18.2	31.2
	55%	4.9	8.7	14.9	30.2	3.7	7.9	14.0	29.2	4.1	8.7	16.0	30.2
	60%	4.4	8.1	13.9	27.6	3.7	6.2	12.8	26.8	3.7	7.9	14.6	27.6
	65%	4.1	7.9	12.9	25.5	3.4	5.9	11.4	24.7	3.3	5.9	13.2	25.5
0.10	50%	5.0	8.4	14.8	28.0	3.8	7.6	14.2	27.2	4.4	8.8	16.3	28.0
	55%	4.4	7.9	13.4	27.1	3.3	7.1	12.6	26.3	3.7	7.9	14.4	27.1
	60%	4.0	7.3	12.5	24.9	3.3	5.6	11.5	24.1	3.3	7.1	13.1	24.9
	65%	3.7	7.1	11.6	23.0	3.0	5.3	10.3	22.3	3.0	5.3	11.9	23.0
0.12	50%	4.1	7.0	12.4	23.4	3.1	6.3	11.8	22.7	3.7	7.4	13.6	23.4
	55%	3.7	6.5	11.2	22.6	2.8	5.9	10.5	21.9	3.1	6.5	12.0	22.6
	60%	3.3	6.1	10.4	20.7	2.7	4.7	9.6	20.1	2.7	5.9	10.9	20.7
	65%	3.1	5.9	9.6	19.1	2.5	4.4	8.5	18.5	2.4	4.4	9.9	19.1
0.14	50%	3.5	6.0	10.6	20.0	2.7	5.4	10.1	19.4	3.2	6.3	11.7	20.0
	55%	3.1	5.6	9.6	19.4	2.4	5.1	9.0	18.8	2.6	5.6	10.3	19.4
	60%	2.8	5.2	8.9	17.7	2.3	4.0	8.2	17.2	2.3	5.1	9.4	17.7
	65%	2.6	5.1	8.2	16.4	2.1	3.8	7.3	15.9	2.1	3.8	8.5	16.4

Notes: Volumes are thousands of vehicles per day.
Assumed values for all entries: 10% trucks, PHF = 0.88, 12-ft lanes, 6-ft shoulders, 10 access points/mi.
Assumed values for Class I—level: BFFS = 65 mi/h, 20% no-passing zones.
Assumed values for Class I—rolling: BFFS = 60 mi/h, 40% no-passing zones.
Assumed values for Class II—rolling: BFFS = 50 mi/h, 60% no-passing zones.

The Class I—level case assumes higher speeds, with significant passing opportunities.

The Class I—rolling case assumes more moderate speeds and reduced passing opportunities because of the terrain.

The Class II—rolling case is similar to a scenic or recreational highway with lower speeds and limited passing opportunities.

A couple of interesting characteristics are displayed in Exhibit 15-46:

a. LOS A is not shown. Even in level terrain, this level can be achieved only at very low demand flow rates (almost always lower than 50 veh/h, directional).

b. The range of demand flows falling within LOS E is broad compared with other levels of service. This is because the quality of service on two-lane highways tends to become unacceptable at relatively low *v/c* ratios. Few two-lane highways are observed operating at or near capacity (except for short segments), because most will have been expanded before demand flows approaching capacity develop.

Exhibit 15-46 should be used only in generalized planning and preliminary engineering analysis. It is best used to examine a number of two-lane highways within a given jurisdiction to determine which need closer scrutiny. If anticipated AADTs on a given segment or facility appear to put the segment or facility into an undesirable LOS, more site-specific data should be obtained (or forecast) and a full operational analysis conducted before any firm commitments to reconstruct or improve the highway are made.

DESIGN ANALYSIS

In design analysis, demand characteristics are generally known. The analysis is intended to give insights into design parameters needed to provide a target LOS for the demand characteristics as stated. For two-lane highways, design decisions are relatively limited. Lane and shoulder widths have a moderate impact on operations but generally do not result in a markedly different LOS.

Typical design projects include horizontal or vertical curve realignments, which may affect percent no-passing zones and free-flow speeds.

The extensions to this chapter's motorized vehicle methodology to consider the impacts of passing lanes and climbing lanes (Section 4) can be used to provide critical design insight. However, the computations are performed in the operational analysis mode, leading to a comparison of operations with or without the passing or climbing lane. Section 4 also describes a number of design treatments for two-lane highways. However, there is no methodology at this point for estimating the impact of these design treatments on operating quality.

Given the relatively few design parameters involved in a two-lane highway, most design analysis is conducted as an iterative series of operational analyses.

PLANNING AND PRELIMINARY ENGINEERING ANALYSIS

Planning and preliminary engineering analysis has the same objectives as design analysis, except that it occurs early in the process when few details of demand and other characteristics are known. Thus, design analysis is augmented by the use of default values for many inputs.

The other principal characteristic of planning and preliminary engineering analysis is that demands are generally described in terms of two-way AADT.

This chapter includes generalized daily service volume tables covering a specific range of default values. They can be used for a coarse and general evaluation of the likely LOS for a two-lane highway in various settings under an expected AADT demand. These tables are useful only for the most preliminary of analyses. For example, all two-lane highway segments in a particular region can be considered by using these criteria. Any segments that appear to be operating at an undesirable LOS should be subjected to site-specific study with a more detailed operational analysis before any major design, reconstruction, or investment decisions are made.

The *Planning and Preliminary Engineering Applications Guide to the HCM* (*18*) provides additional guidance on adapting this chapter's default values, performance measures, and methods to planning and preliminary engineering analyses.

USE OF ALTERNATIVE TOOLS

As noted previously in Section 3, no alternative deterministic tools are in common use for two-lane highway analysis (*5*).

7. REFERENCES

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

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VOLUME 2 INDEX

The index to Volume 2 lists the text citations of the terms defined in the Glossary (Volume 1, Chapter 9). Volumes 1, 2, and 3 are separately indexed. In the index listings, the first number in each hyphenated pair of numbers indicates the chapter, and the number after the hyphen indicates the page within the chapter.

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