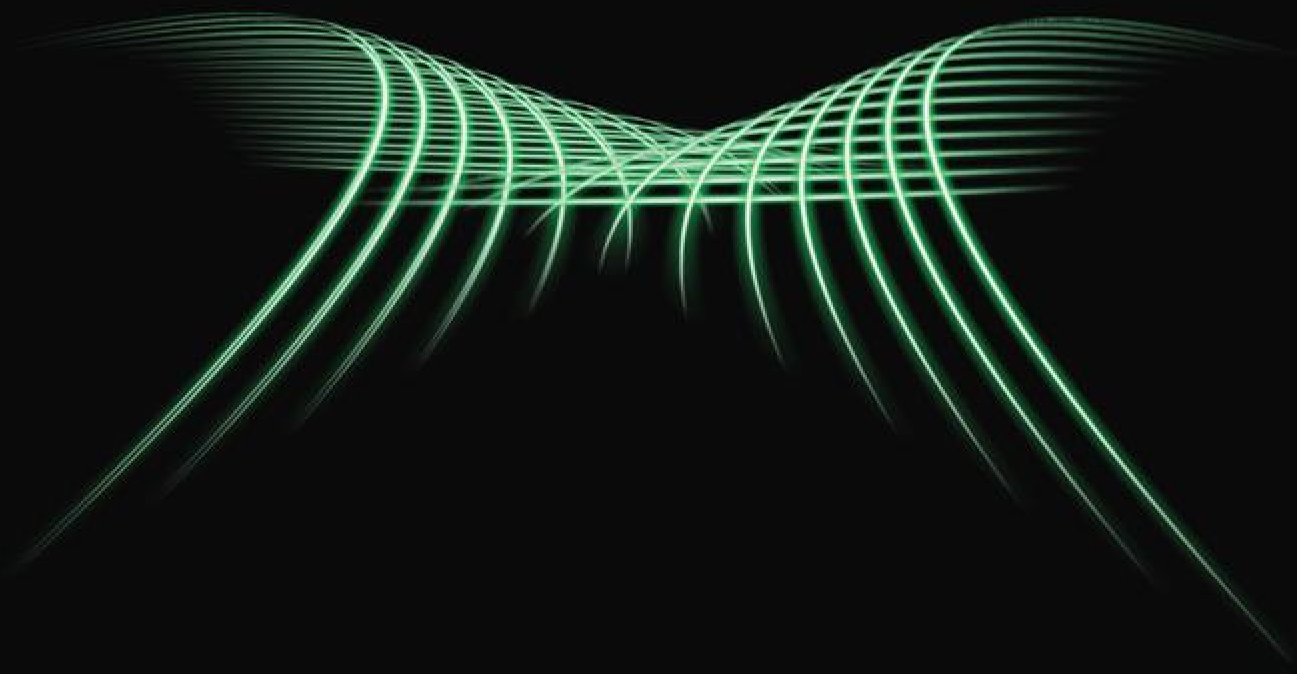
HCM2010

HIGHWAY CAPACITY MANUAL



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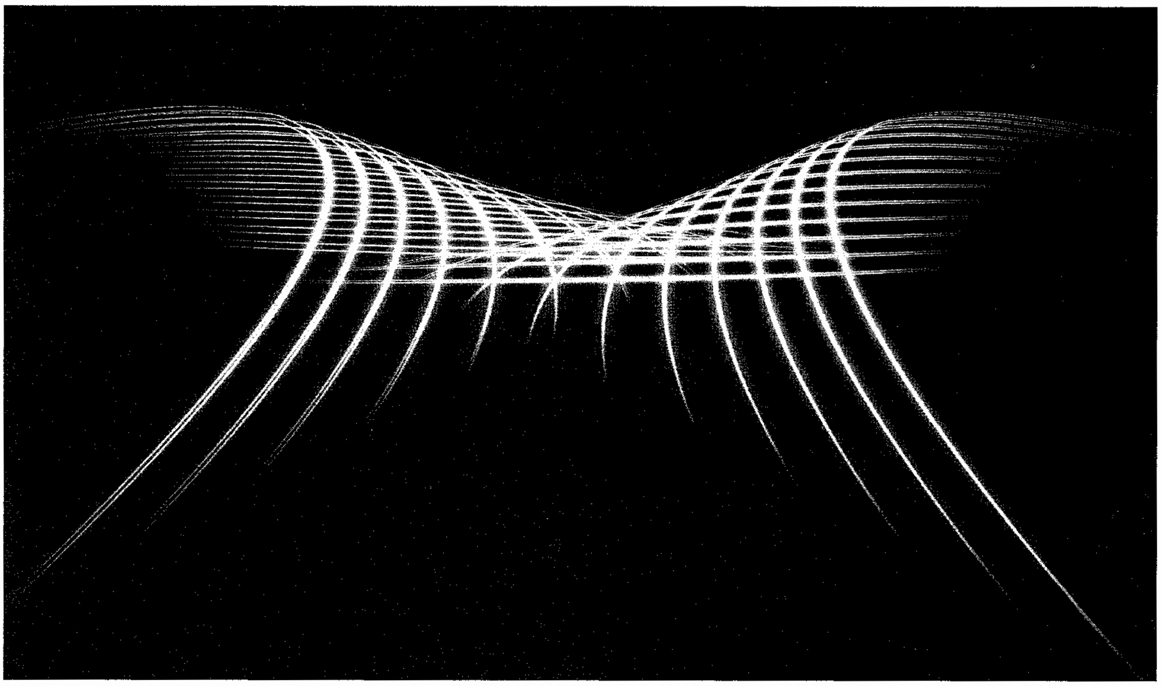

VOLUME 2: UNINTERRUPTED FLOW

TRB TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

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VOLUME 2
UNINTERRUPTED FLOW

OVERVIEW

Volume 2 of the *Highway Capacity Manual* (HCM) contains six chapters that present analysis methods for uninterrupted-flow roadways—that is, roadways that have no fixed causes of delay or interruption external to the traffic stream. This volume addresses three types of uninterrupted-flow roadways:

- *Freeways,* defined as separated highways with full control of access and two or more lanes in each direction dedicated to the exclusive use of traffic;
- *Multilane highways,* defined as highways that do not have full control of access and that have two or more lanes in each direction, with traffic signals or roundabouts spaced at least 2 mi apart on average; and
- *Two-lane highways,* defined as roadways with one lane for traffic in each direction (except for occasional passing lanes or truck climbing lanes), with traffic signals, roundabouts, or STOP-controlled intersections spaced at least 2 mi apart on average.

The HCM treats roadways that have traffic signals, roundabouts, or STOP-controlled intersections spaced less than 2 mi apart on average as urban streets. Urban streets are discussed in Volume 3, Interrupted Flow.

VOLUME ORGANIZATION

Freeways

Traffic enters and exits a freeway via ramps. **Chapter 13, Freeway Merge and Diverge Segments,** focuses on locations where two or more traffic streams combine to form a single traffic stream (a *merge*) or where a single traffic stream divides to form two or more separate traffic streams (a *diverge*). These locations are most commonly ramp–freeway junctions but include points where mainline roadways join or separate. Chapter 13 can also be applied in an approximate way to ramp–highway junctions on multilane highways and collector–distributor roads. Ramp–street junctions are analyzed with the methods in the intersection and interchange chapters in Volume 3.

Sometimes freeway merges are closely followed by freeway diverges, or a one-lane off-ramp closely follows a one-lane on-ramp and the two are connected by a continuous auxiliary lane. In these cases, the traffic streams to and from the ramps must cross each other over a significant length of freeway without the aid of traffic control devices (except for guide signs). The term "closely" implies that the distance between the merge and diverge segments is not sufficient for them to operate independently, thus creating a *weave.* **Chapter 12, Freeway Weaving Segments,** provides procedures for analyzing weaving operations on freeways.

It can be applied in an approximate way to weaves on multilane highways and collector–distributor roads, but not to weaves on arterial streets.

The remaining portions of the freeway mainline that are not merge, diverge, or weaving segments (except for toll plazas, drawbridges, or similar points where freeway traffic may be temporarily required to stop) are covered in **Chapter 11, Basic Freeway Segments.** This chapter also provides information on the base conditions and passenger car equivalents for heavy vehicles that are common to all of the freeway chapters.

Chapter 10, Freeway Facilities, provides a 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, the term *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.

The methodologies of Chapters 11, 12, and 13 all focus on a single time period of interest, generally the peak 15 min within a peak hour. However, Chapter 10's methodology allows for the analysis of multiple and continuous 15-min periods and is capable of identifying breakdowns and the impact of such breakdowns over space and time.

Multilane Highways

Chapter 14, Multilane Highways, presents analysis methods for the portions of multilane highways away from the influence of signalized intersections (or other forms of intersection traffic control that interrupt the flow of traffic on the highway). 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 signalized intersections are analyzed with the Chapter 14 methodology. Isolated signalized intersections should be analyzed with the methodology of Chapter 18, Signalized Intersections.

Bicycles are typically permitted on multilane highways, and multilane highways often serve as primary routes for both commuter cyclists (on suburban highways) and recreational cyclists (on rural highways). Chapter 14 presents a method for estimating the bicycle level of service (LOS) on multilane highways.

Two-Lane Highways

Chapter 15, Two-Lane Highways, presents analysis methods for the portions of two-lane highways that are away from the influence of intersection traffic control that interrupts the flow of traffic. In general, any segment that is 2.0 to 3.0 mi from the nearest signalized intersection, roundabout, or intersection where the highway is STOP-controlled would fit into this category. Where these interruptions to traffic 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 17, Urban Street Segments, which are located in Volume 3.

Chapter 15 can be used to analyze three classes of two-lane highways:

- *Class I* highways are ones where motorists expect to travel at relatively high speeds, such as major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks;
- *Class II* highways are ones where motorists do not necessarily expect to travel at high speeds, such as highways serving as access routes to Class I facilities, serving as scenic or recreational routes, or passing through rugged terrain; and
- *Class III* highways are ones serving moderately developed areas, such as portions of a Class I or Class II highway passing through small towns or developed recreational areas or longer segments passing through more spread-out recreational areas, with increased roadside densities.

Two-lane highways often serve as routes for recreational cyclists. Chapter 15 presents a method for estimating the bicycle LOS on these highways.

RELATED CHAPTERS

Volume 1

The chapters in Volume 2 assume that the reader is already familiar with the concepts presented in the Volume 1 chapters, in particular the following:

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

Volume 3

The intersection and interchange chapters (Chapters 18–22) are used to determine the operations of freeway ramp–street junctions and the operations of isolated traffic signals, roundabouts, and STOP-controlled intersections along multilane and two-lane highways. In the context of Volume 2, it is particularly important to examine the length of the queue extending back from a freeway off-ramp–street junction, since long queues may affect freeway operations, a situation that is not accounted for in the HCM techniques.

VOLUME 4: APPLICATIONS GUIDE
Methodological Details
25. Freeway Facilities: Supplemental
26. Freeway and Highway Segments: Supplemental
27. Freeway Weaving: Supplemental
28. Freeway Merges and Diverges: Supplemental
35. Active Traffic Management
Case Studies
Technical Reference Library

Volume 4

Five chapters in Volume 4 (accessible at www.HCM2010.org) provide additional information that supplements the material presented in Volume 2. These chapters are as follows:

- *Chapter 25, Freeway Facilities: Supplemental*—details of the computations used in the Chapter 10 methodology, and computational engine flowcharts and linkage lists;
- *Chapter 26, Freeway and Highway Segments: Supplemental*—examples of applying alternative tools to situations that are not addressed by the Chapter 11 method for basic freeway segments, and state-specific default values for heavy vehicle percentage that apply to all Volume 2 chapters;
- *Chapter 27, Freeway Weaving: Supplemental*—examples of applying alternative tools to situations not addressed by the Chapter 12 method;
- *Chapter 28, Freeway Merges and Diverges: Supplemental*—examples of applying alternative tools to situations not addressed by the Chapter 13 method; and
- *Chapter 35, Active Traffic Management*—descriptions of active traffic management strategies; a discussion of the mechanisms by which they affect demand, capacity, and performance; and general guidance on possible evaluation methods for active traffic management techniques.

Access Volume 4 at www.HCM2010.org

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

- *Case Study No. 3* illustrates the process of applying HCM techniques to the analysis of a two-lane highway;
- *Case Study No. 4* illustrates the process of applying HCM techniques to the analysis of a freeway; and
- *Case Study No. 6* illustrates the application of alternative tools to a freeway facility in a situation where HCM techniques are unsuitable.

Case Studies No. 3 and No. 4 focus on the process of applying the HCM rather than on the details of performing calculations (which are addressed by the example problems in the Volume 2 chapters). These case studies' computational results were developed by using HCM2000 methodologies and therefore may not match the results obtained from applying the HCM 2010. However, the process of application is the focus, not the specific computational results.

The Technical Reference Library in Volume 4 contains copies of (or links to) many of the documents referenced in Volume 2 and its supplemental chapters. Because the Chapter 10 methodology is too complex to be implemented by manual pencil-and-paper techniques, the FREEVAL-2010 spreadsheet has been developed to implement the methodology's calculations. The Technical Reference Library contains a copy of the spreadsheet along with a user's guide.

LEVELS OF ANALYSIS AND ANALYSIS TOOLS

As discussed in Chapter 2, Applications, HCM methodologies can be applied to the operations, design, preliminary engineering, and planning levels of analysis. These levels differ both in the amount of field data used in the analysis (as opposed to default values) and in the way the HCM is applied (iteratively, to find a design that meets a desired set of criteria, or as a single application, to evaluate performance given a particular set of inputs). Each Volume 2 chapter provides a section that discusses how to apply the chapter to these different levels of analysis, along with a section with recommended default values for planning and preliminary engineering analyses.

Three Volume 2 chapters (10, 14, and 15) provide generalized service volume tables applicable to freeway facilities, multilane highways, and two-lane highways, respectively. These tables can be used for large-scale planning efforts when the goal is to analyze a large number of facilities to determine where problems might exist or arise or where improvements might be needed. Any facilities identified as likely to experience problems or need improvement should then be subjected to a more detailed analysis that takes into account the existing or likely future characteristics of the specific facility before any detailed decisions on implementing specific improvements are made. Because the service volumes provided in these tables are highly dependent on the default values assumed as inputs, it is recommended that users wishing to apply generalized service volume tables develop their own tables by using local default values, in accordance with the processes described in Appendix A and Appendix B of Chapter 6, HCM and Alternative Analysis Tools.

Chapter 6 also describes in general terms the conditions under which the use of alternative tools to supplement HCM capacity and quality-of-service procedures should be considered. Each Volume 2 chapter contains a section discussing the potential application of alternative tools to the specific system element addressed by the chapter, and Chapters 26–28 in Volume 4 provide example problems illustrating applications of alternative tools to address HCM limitations. Each chapter lists the specific limitations of its methodology. The major limitations are summarized as follows:

- *Freeways*
  - Operations of oversaturated freeway segments (but not necessarily oversaturated freeway facilities, as discussed later)
  - Multiple overlapping breakdowns or bottlenecks
  - Conditions where off-ramp queues extend back onto the freeway or affect the behavior of exiting vehicles
  - Operation of separated high-occupancy vehicle (HOV) facilities and weaving interactions between HOV and general-purpose lanes
  - Toll plaza operations
  - Ramp-metering effects

- *Multilane highways*
    - Operations during oversaturated conditions
    - The impacts of shoulder parking activity, bus stops, or significant pedestrian activity
    - Possible queuing impacts when a multilane highway segment transitions to a two-lane highway segment
    - Differences between various types of median barriers, and the difference between the impact of a median barrier and a two-way left-turn lane
    - The range of values used to develop the bicycle LOS model (although the 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)
- *Two-lane highways*
    - Operations during oversaturated conditions
    - Impact of intersection traffic control on the overall facility LOS
    - The range of values used to develop the bicycle LOS model (although the model has been successfully applied to rural two-lane highways, users should be aware that conditions on many of those highways are outside the range of values used to develop the model)

If an analysis of an individual freeway segment reveals the segment to be oversaturated, then Chapter 10, Freeway Facilities, must be used to assess operation of the segment and its impacts on upstream and downstream sections. If the Chapter 10 analysis reveals that the oversaturation would extend beyond the geographic or temporal boundaries of the analysis, then the boundaries of the Chapter 10 analysis should be expanded to contain the oversaturation. If expanding the boundaries of the analysis is not practical, then no analytical tool, including the HCM, can give a complete answer in this situation.

CHAPTER 10
FREEWAY FACILITIES

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

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

- *Freeway merge and diverge segments:* 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).
- *Freeway weaving segments:* 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.
- *Basic freeway segments:* All segments that are not merge, diverge, or weaving segments.

Analysis methodologies are detailed for basic freeway segments in Chapter 11, for weaving segments in Chapter 12, and for merge and diverge segments in Chapter 13.

Chapter 10, Freeway Facilities, provides a 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, the term *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.

The methodologies of Chapters 11, 12, and 13 focus on a single time period of interest, generally the peak 15 min within a peak hour. This chapter's methodology allows for the analysis of multiple and continuous 15-min time periods and is capable of identifying breakdowns and the impact of such breakdowns over space and time.

The methodology is integral with the FREEVAL-2010 model, which implements the complex computations involved. This chapter discusses the basic principles of the methodology and its application. Chapter 25, Freeway Facilities: Supplemental, provides a complete and detailed description of all the algorithms that define the methodology. The Technical Reference Library in Volume 4 contains a user's guide to FREEVAL-2010 and an executable spreadsheet that implements the methodology.

SEGMENTS AND INFLUENCE AREAS

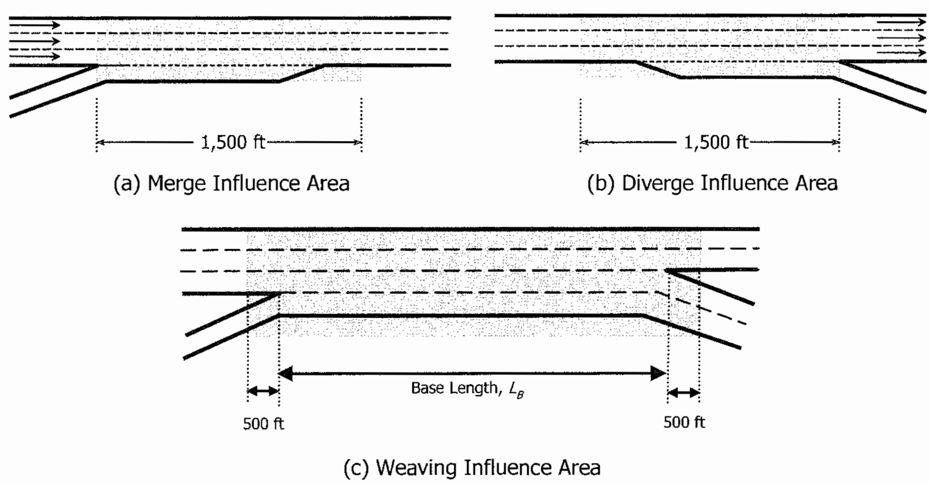
It is important that the definition of freeway segments and their influence areas be clearly understood. The influence areas of merge, diverge, and weaving segments are as follows:

- *Weaving segment:* 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.
- *Merge segment:* 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.
- *Diverge segment:* From the point where the edges of the travel lanes of the merging 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.

Exhibit 10-1
Influence Areas of Merge, Diverge, and Weaving Segments



Basic freeway segments are any other segments along the freeway that are not within these defined influence areas, which is not to say that basic freeway segments are not affected by the presence of adjacent and nearby merge, diverge, and weaving segments. Particularly when a segment breaks down, its effects will propagate to both upstream and downstream segments, regardless of type. Furthermore, the general 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, do exist even on urban freeways where merge and diverge points (most often ramps) are closely spaced. Exhibit 10-2 illustrates this point. It shows a 9,100-ft (1.7-mi) length of freeway with four ramp terminals, two of which form a weaving segment. Even with an average ramp spacing 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 it is true that 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.

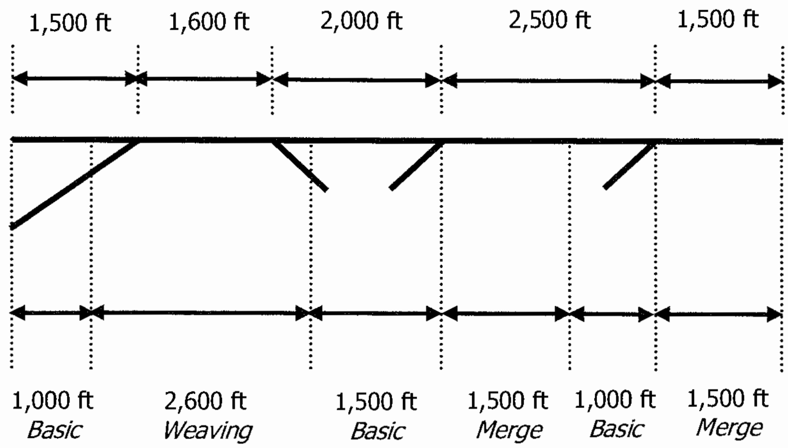


Exhibit 10-2
Basic Freeway Segments on an Urban Freeway

FREE-FLOW SPEED

Free-flow speed is strictly defined as the theoretical speed when the density and flow rate on the study segment are both zero. Chapter 11, Basic Freeway Segments, presents speed–flow curves that indicate that the free-flow speed on freeways is expected to prevail at flow rates between 0 and 1,000 passenger cars per hour per lane (pc/h/ln). In this broad range of flows, speed is insensitive to flow rates. This characteristic simplifies and allows for measurement of free-flow speeds in the field.

Chapter 11 also presents a methodology for estimating the free-flow speed of a basic freeway segment if it cannot be directly measured. The free-flow speed of a basic freeway segment is sensitive to three variables:

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

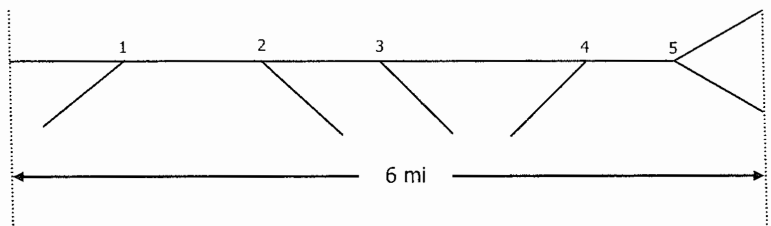
The most critical of these variables is total ramp density. *Total ramp density* is defined as the average number of on-ramp, off-ramp, major merge, and major diverge junctions per mile. It applies to a 6-mi segment of freeway facility, 3 mi upstream and 3 mi downstream of the midpoint of the study segment.

While the methodology for determining free-flow speed is provided in Chapter 11, Basic Freeway Segments, it is also applied in Chapter 12, Freeway Weaving Segments, and Chapter 13, Freeway Merge and Diverge Segments. Thus, free-flow speed affects the operation of all basic, weaving, merge, and diverge segments on a freeway facility.

The free-flow speed is an important characteristic, as the capacity *c*, service flow rates *SF*, service volumes *SV*, and daily service volumes *DSV* all depend on it.

Exhibit 10-3 illustrates the determination of total ramp density on a 6-mi length of freeway facility.

Exhibit 10-3
Ramp Density Determination



As illustrated in Exhibit 10-3, there are four ramp terminals and one major diverge point in the 6-mi segment illustrated. The total ramp density is, therefore, 5/6 = 0.83 ramp/mi.

CAPACITY OF FREEWAY FACILITIES

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

The methodologies of Chapters 11, 12, and 13 allow the capacity of each basic freeway, freeway weaving, freeway merge, and freeway diverge segment to be estimated. It is highly unlikely that every segment of a facility will have the same roadway, traffic, and control conditions and even less likely that they will have the same capacity.

Conceptual Approach to the Capacity of a Freeway Facility

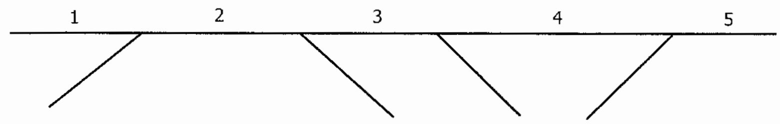
Consider the example shown in Exhibit 10-4. It illustrates five consecutive segments that are to be analyzed as one "freeway facility." Demand flow rates v_d, capacities c, and actual flow rates v_a are shown, as are the resulting v_d/c and v_a/c ratios. A lane is added in Segment 3 (even though this segment begins with an off-ramp), providing higher capacities for Segments 3, 4, and 5 than in Segments 1 and 2. The example analyzes three scenarios.

In Scenario 1, none of the demand flow rates exceeds the capacities of the segments that make up the facility. Thus, no breakdowns occur, and the actual flow rates are the same as the demand flow rates (i.e., $v_d = v_a$ for this scenario). None of the v_d/c or v_a/c ratios exceeds 1.00, although the highest ratios (0.978) occur in Segment 5.

Scenario 2 adds 200 vehicles per hour (veh/h) of demand to each segment (essentially another 200 veh/h of through freeway vehicles). In this case, Segment 5 will experience a breakdown—that is, the demand flow rate will exceed the capacity. In this segment, demand flow rate v_d differs from the actual flow rate v_a, as the actual flow rate v_a can never exceed the capacity c.

In Scenario 3, all demand flow rates are increased by 10%, which, in effect, keeps the relative values of the segment demand flow rates constant. In this case, demand flow rate will exceed capacity in Segments 4 and 5. Again, the demand flow rates and actual flow rates will differ in these segments.

Exhibit 10-4
Example of the Effect of Segment Capacity on a Freeway Facility



Scenario	Performance Measures	Freeway Segment 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 segment)	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 demand by 10% in all segments)	Demand v_d, veh/h	3,740	3,850	3,740	4,840	5,060
	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	1.078	1.120
	v_a/c ratio	0.935	0.963	0.831	1.000	1.000

Note: Shaded cells indicate segments where demand exceeds capacity.

This example highlights a number of points that make the analysis of freeway facilities very complicated:

1. It is critical to this methodology 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.
2. In Scenarios 2 and 3, the analysis of Exhibit 10-4 is inadequate and misleading. In Scenario 2, when Segment 5 breaks down, queues begin to form and to propagate upstream. Thus, even though the demands in Segments 1 through 4 are less than the capacity of those segments, the queues generated by Segment 5 over time will propagate through Segments 1 through 4 and significantly affect their operation. In Scenario 3, Segments 4 and 5 fail, and queues are generated, which also propagate upstream over time.
3. It might be argued that the analysis of Scenario 1 is sufficient to understand the facility operation as long as all segments are undersaturated (i.e., all segment v_d/c ratios are less than or equal to 1.00). However, when any segment v_d/c ratio exceeds 1.00, such a simple analysis ignores the spreading impact of breakdowns in space and time.
4. In Scenarios 2 and 3, the segments downstream of Segment 5 will also be affected, as demand flow is prevented from reaching those segments by the Segment 5 (and Segment 4 in Scenario 3) breakdowns and queues.
5. In this example, it is also important to note that the segment(s) that break down first do not have the lowest capacities. Segments 1 and 2, with lower capacities, do not break down in any of the scenarios. Breakdown occurs first in Segment 5, which has one of the higher capacities.

Considering all these complications, the capacity of a freeway facility is defined as follows:

Freeway facility capacity is the capacity of the critical segment among those segments composing the defined facility. This capacity must, for analysis purposes, be compared with the demand flow rate on the critical segment.

The *critical segment* is defined as the segment that will break down first, 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 can change over time as the demand pattern changes. Facility capacity may be more than the capacity of the component segment with the lowest capacity. Therefore, it is important that individual segment demands and capacities be evaluated. The fact that one of these segments will be the critical one and will define the facility capacity does not diminish the importance of the capacities of other segments in the defined facility.

Base Capacity of Freeway Facilities

In the methodologies of Chapters 11, 12, and 13, a base capacity is used. The base capacity represents the capacity of the facility, assuming that there are no heavy vehicles in the traffic stream and that all drivers are regular users of the segment. The base capacity for all freeway segments varies with the free-flow speed, as indicated in Exhibit 10-5.

Exhibit 10-5
Free-Flow Speed vs. Base Capacity for Freeways

Free-Flow Speed (mi/h)	Base Capacity (pc/h/ln)
75	2,400
70	2,400
65	2,350
60	2,300
55	2,250

The equation given in Chapter 11, Basic Freeway Segments, for estimating the free-flow speed of a segment is as shown in Equation 10-1:

Equation 10-1

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\ TRD^{0.84}$$

where

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

f_{LW} = adjustment for lane width (mi/h),

f_{LC} = adjustment for lateral clearance (mi/h), and

TRD = total ramp density (ramps/mi).

The process for determining the value of adjustment factors is described in Chapter 11.

Because the base capacity of a freeway segment is directly related to the free-flow speed, it is possible to construct a relationship between base capacity and the lane width, lateral clearance, and total ramp density of the segment. If the lane width and lateral clearance are taken to be their base values (12 and 6 ft, respectively), a relationship between base capacity and total ramp density emerges, as shown in Exhibit 10-6.

Base capacity is expressed as a flow rate for a 15-min analysis period, not a full-hour volume. It also represents a flow rate in pc/h, with no heavy vehicles, and a driver population familiar with the characteristics of the analysis segment.

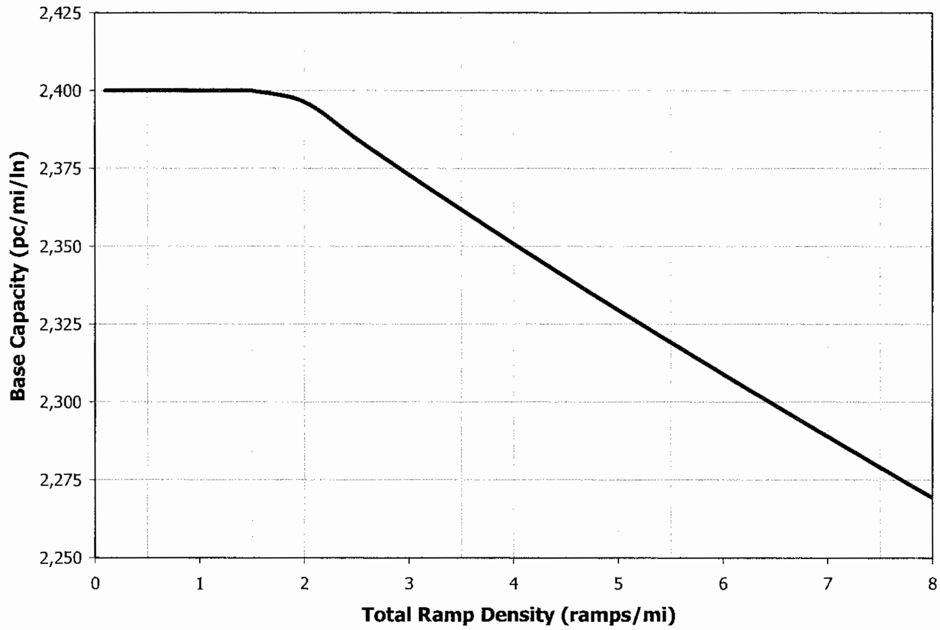


Exhibit 10-6
Base Capacity vs. Total Ramp Density

Segment Capacity vs. Facility Capacity

Free-flow speed is a characteristic of a length of freeway extending 3 mi upstream and 3 mi downstream of the center point of an analysis segment. The segment may be a basic freeway segment, a weaving segment, a merge segment, or a diverge segment. In essence, it is a measure of the impact of overall facility characteristics on the operation of the individual analysis segment centered in the defined 6-mi range.

This concept can be somewhat generalized where freeway facility analysis is involved. If conditions (particularly ramp density) are similar along a greater length of freeway, it is acceptable to compute the total ramp density for the greater length and apply it to all segments within the analysis length. This process assumes that moving the "center" of a 6-mi length for each component segment will not result in a significant change in the free-flow speed.

The capacity of a nearly homogeneous freeway facility is, for all practical purposes, the same as the capacity of a basic freeway segment with the same roadway and traffic characteristics. Consider the following:

- Merge and diverge segments have the same capacity as a similar basic freeway segment. As discussed in Chapter 13, the presence of merge and diverge segments on a freeway may affect operating characteristics, generally reducing speeds and increasing densities, but does not reduce capacity.
- Weaving segments often have per lane capacities that are less than those of the entering and leaving basic freeway segments. In almost all cases, however, weaving segments have more lanes than the entering and

leaving basic freeway segments. Thus, the impact on the capacity of the mainline freeway most often is negligible.

This does not mean, however, that the capacity of each component segment of a facility is the same. Each segment has its own demand and demand characteristics. Demand flow rate can change at every entry and exit point along the freeway, and the percent of heavy vehicles can change too. Terrain also can change at various points along the freeway.

Changes in heavy vehicle presence can change the capacity of individual segments within a defined facility. Changes in the split of movements in a weaving segment can change its capacity. In the same way, changes in the relative demand flows at on- and off-ramps can change the location of the critical segment within a defined facility and its capacity.

As noted previously, the capacity of a freeway facility is defined as the capacity of its critical segment.

LOS: COMPONENT SEGMENTS AND THE FREEWAY FACILITY

LOS of Component Segments

Chapters 11, 12, and 13 provide methodologies to determine the LOS in basic, weaving, merge, and diverge segments. In all cases, LOS F is identified when v_d/c is greater than 1.00. Such breakdowns are easily identified, and users are referred to this chapter.

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 from a downstream breakdown extends into an upstream segment. The latter cannot be done by using the individual segment analysis procedures of Chapters 11, 12, and 13.

Thus, when facility-level analysis is undertaken by using the methodology of this chapter, LOS F for a component segment will be identified in two different ways:

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

The latter 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.

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 the number of lanes in each segment, as shown in Equation 10-2:

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

Equation 10-2

where

D_F = average density for the facility (pc/mi/ln),

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

L_i = length of segment i (ft),

N_i = number of lanes in segment i, and

n = number of segments in the defined facility.

The LOS criteria for a freeway facility are shown in Exhibit 10-7. They are the same criteria used for basic freeway segments.

Exhibit 10-7
LOS Criteria for Freeway Facilities

Level of Service	Density (pc/mi/ln)
A	≤11
B	>11–18
C	>18–26
D	>26–35
E	>35–45
F	>45 or any component v_d/c ratio > 1.00

Use of a LOS descriptor for the overall freeway facility must be done with care. It is critical that the LOS for individual segments composing the facility also be reported. Because the overall LOS is an average, it 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 exist in one or more segments of a facility, the average LOS is of limited use. The average LOS applies to a specific time period, usually 15 min.

While LOS A through D are defined by using the same densities that apply to basic freeway segments, LOS F for a facility is defined as a case in which any component segment of the freeway exceeds a v_d/c ratio of 1.00 or the average density over the defined facility exceeds 45 pc/mi/ln. In such a case, this chapter's methodology allows the analyst to map the impacts of this breakdown in time and space, and close attention to the individual LOS of component segments is necessary.

SERVICE FLOW RATES, SERVICE VOLUMES, AND DAILY SERVICE VOLUMES FOR A FREEWAY FACILITY

Just as each segment of a freeway facility has its own capacity, each segment also has a set of service flow rates SF_i for each LOS. A service flow rate is the maximum directional rate of flow that can be sustained in a given segment without violating the criteria for LOS i. Service flow rates are stated in vehicles per hour under prevailing roadway, traffic, and control conditions. By definition, the service flow rate for LOS E is synonymous with capacity for all uninterrupted-flow facilities and their component segments.

Chapters 11, 12, and 13 provide complete discussions of how to determine service flow rates for basic, weaving, merge, and diverge freeway segments.

A service volume SV_i is the maximum hourly directional volume that can be sustained in a given segment without violating the criteria for LOS i during the worst 15 min of the hour (period with the highest density) under prevailing roadway, traffic, and control conditions. Once a set of service flow rates has been established for a segment, the service volume is found from Equation 10-3:

Equation 10-3

$$SV_i = SF_i \times PHF$$

where

SV_i = service volume for LOS i (veh/h),

SF_i = service flow rate for LOS i (veh/h), and

PHF = peak hour factor.

A daily service volume DSV_i is the maximum total daily volume in both directions that can be sustained in a given segment without violating the criteria for LOS i in the peak direction in the worst 15 min of the peak hour under prevailing roadway, traffic, and control conditions. Given a set of service volumes for a segment, the daily service volume is found from Equation 10-4:

Equation 10-4

$$DSV_i = \frac{SV_i}{K \times D}$$

where

DSV_i = daily service volume (veh/day),

K = proportion of daily traffic occurring in the peak hour of the day, and

D = proportion of traffic in the peak direction during the peak hour of the day.

The capacity of a freeway facility has been defined as the capacity (under prevailing conditions) of the critical segment. For consistency, therefore, other service flow rates must also be applied to the critical segment.

For an overall understanding of the freeway facility, the LOS and service flow rates (or service volumes or daily service volumes) of the individual component segments must be considered along with the overall average LOS for the defined facility and its service flow rate.

GENERALIZED DAILY SERVICE VOLUMES FOR FREEWAY FACILITIES

Generalized daily service volume tables provide a means to assess all freeways in a region or jurisdiction quickly to determine which segments need to be assessed more carefully (using operational analysis) to ameliorate existing or pending problems.

To generate a generalized daily service volume table for freeway facilities, several simplifying assumptions must be made. The assumptions made here include the following:

1. All segments of the freeway have the same basic number of lanes (two, three, or four in each direction).
2. Lane widths are 12 ft, and lateral clearances are 6 ft.
3. All on-ramps and off-ramps handle the same percentage of freeway traffic. This setup maintains a reasonably consistent demand flow rate on each segment of the facility.
4. The first ramp on the defined freeway facility is an off-ramp. This assumption is necessary to implement Item 5, below.
5. Given the demand characteristics of Items 2 and 3, all daily service volumes are stated in terms of the demand *entering* the defined freeway facility at its upstream boundary.
6. The terrain is the same in all segments of the facility.
7. The heavy vehicle percentage is the same in all segments of the facility.

On the basis of these assumptions, generalized daily service-volume tables are shown in Exhibit 10-8 (for urban freeways) and Exhibit 10-9 (for rural freeways).

Generalized service volumes are provided for level and rolling terrain; for four-lane, six-lane, and eight-lane freeways (both directions); and for a variety of combinations of the *K*-factor and *D*-factor. To use the table, analysts must select a combination of *K* and *D* appropriate for their state or region. Additional assumptions made for urban and rural freeways are listed here.

Assumptions for urban freeways:

- Total ramp density = 3.00 ramps/mi (i.e., ⅓-mi average spacing between ramps);
- 5% trucks, no recreational vehicles (RVs), and no buses;
- $PHF = 0.95$; and
- $f_p = 1.00$.

Assumptions for rural freeways:

- Total ramp density = 0.20 ramp/mi (i.e., 5-mi average spacing between ramps);
- 12% trucks, no RVs, and no buses;
- $PHF = 0.88$; and
- $f_p = 0.85$.

Generalized daily service volumes are based on the maximum service flow rate values for basic freeway segments. Exhibit 11-17 (Chapter 11) shows maximum service flow rates *MSF* for basic freeway segments. They are converted to service flow rates under prevailing conditions by multiplying by the number of lanes in one direction *N*, the heavy-vehicle adjustment factor f_{HV}, and the driver-population adjustment factor f_p. Equation 10-3 and Equation 10-4 are then used to convert the service flow rate *SF* to a service volume *SV* and a daily service volume *DSV*.

By combining these equations, the daily service volumes *DSV* of Exhibit 10-8 and Exhibit 10-9 are estimated from Equation 10-5:

Equation 10-5

$$DSV_i = \frac{MSF_i \times N \times f_{HV} \times f_p \times PHF}{K \times D}$$

where all variables are as previously defined.

In applying Equation 10-5, the values of *MSF* are selected from Exhibit 11-17 (Chapter 11), and values for the heavy vehicle and driver population adjustment factors are computed in accordance with the methodology of Chapter 11. The *MSF* for LOS E, which is capacity, may be taken directly from Exhibit 10-5, based on the total ramp density, as lane widths and lateral clearances are standard and have no effect on the *FFS* and thus no effect on the resulting capacities.

Exhibit 10-8 and Exhibit 10-9 are provided for general planning use and should *not* be used to analyze any specific freeway or to make final decisions on important design features. A full operational analysis using this chapter's methodology is required for such specific applications.

The exhibits are useful, however, in evaluating the overall performance of many freeways within a jurisdiction, as a first pass in determining where problems might exist or arise, and in deciding where improvements might be needed. Any freeways identified as likely to experience problems or to need improvement, however, should be subjected to a full operational analysis before any detailed decisions on implementing specific improvements are made.

Daily service volumes are heavily affected by the *K*- and *D*-factors chosen as typical for the analysis. It is important that the analyst use values that are reasonable for the facilities under study. Also, if any characteristic differs significantly from the typical values used to develop Exhibit 10-8 and Exhibit 10-9, the values taken from these exhibits will not be representative of the study facilities.

Exhibit 10-8
Generalized Daily Service Volumes for Urban Freeway Facilities (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Freeways				Six-Lane Freeways				Eight-Lane Freeways			
		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	54.2	75.5	94.1	108.9	81.3	113.3	141.1	163.4	108.4	151.1	188.1	217.8
	0.55	49.3	68.7	85.5	99.0	73.9	103.0	128.3	148.5	98.6	137.3	171.0	198.0
	0.60	45.2	62.9	78.4	90.8	67.8	94.4	117.6	136.1	90.4	125.9	156.8	181.5
	0.65	41.7	58.1	72.4	83.8	62.6	87.2	108.5	125.7	83.4	116.2	144.7	167.5
0.09	0.50	48.2	67.1	83.6	96.8	72.3	100.7	125.4	145.2	96.4	134.3	167.2	193.6
	0.55	43.8	61.0	76.0	88.0	65.7	91.6	114.0	132.0	87.6	122.1	152.0	176.0
	0.60	40.2	56.0	69.7	80.7	60.2	83.9	104.5	121.0	80.3	111.9	139.4	161.3
	0.65	37.1	51.6	64.3	74.5	55.6	77.5	96.5	111.7	74.1	103.3	128.6	148.9
0.10	0.50	43.4	60.4	75.3	87.1	65.1	90.6	112.9	130.7	86.8	120.9	150.5	174.2
	0.55	39.4	54.9	68.4	79.2	59.1	82.4	102.6	118.8	78.9	109.9	136.8	158.4
	0.60	36.1	50.4	62.7	72.6	54.2	75.5	94.1	108.9	72.3	100.7	125.4	145.2
	0.65	33.4	46.5	57.9	67.0	50.0	69.7	86.8	100.5	66.7	93.0	115.8	134.0
0.11	0.50	39.4	54.9	68.4	79.2	59.1	82.4	102.6	118.8	78.9	109.9	136.8	158.4
	0.55	35.8	49.9	62.2	72.0	53.8	74.9	93.3	108.0	71.7	99.9	124.4	144.0
	0.60	32.9	45.8	57.0	66.0	49.3	68.7	85.5	99.0	65.7	91.6	114.0	132.0
	0.65	30.3	42.3	52.6	60.9	45.5	63.4	78.9	91.4	60.7	84.5	105.3	121.8
Rolling Terrain													
0.08	0.50	51.7	72.0	89.7	103.8	77.5	108.0	134.5	155.8	103.4	144.0	179.4	207.7
	0.55	47.0	65.5	81.5	94.4	70.5	98.2	122.3	141.6	94.0	131.0	163.1	188.8
	0.60	43.1	60.0	74.7	86.5	64.6	90.0	112.1	129.8	86.2	120.0	149.5	173.1
	0.65	39.8	55.4	69.0	79.9	59.7	83.1	103.5	119.8	79.5	110.8	138.0	159.7
0.09	0.50	46.0	64.0	79.7	92.3	68.9	96.0	119.6	138.4	91.9	128.0	159.5	184.6
	0.55	41.8	58.2	72.5	83.9	62.7	87.3	108.7	125.9	83.6	116.4	145.0	167.8
	0.60	38.3	53.4	66.4	76.9	57.4	80.0	99.7	115.4	76.6	106.7	132.9	153.8
	0.65	35.3	49.2	61.3	71.0	53.0	73.9	92.0	106.5	70.7	98.5	122.7	142.0
0.10	0.50	41.4	57.6	71.8	83.1	62.0	86.4	107.6	124.6	82.7	115.2	143.5	166.1
	0.55	37.6	52.4	65.2	75.5	56.4	78.6	97.9	113.3	75.2	104.8	130.5	151.0
	0.60	34.5	48.0	59.8	69.2	51.7	72.0	89.7	103.8	68.9	96.0	119.6	138.4
	0.65	31.8	44.3	55.2	63.9	47.7	66.5	82.8	95.8	63.6	88.6	110.4	127.8
0.11	0.50	37.6	52.4	65.2	75.5	56.4	78.6	97.9	113.3	75.2	104.8	130.5	151.0
	0.55	34.2	47.6	59.3	68.7	51.3	71.4	89.0	103.0	68.4	95.2	118.6	137.3
	0.60	31.3	43.7	54.4	62.9	47.0	65.5	81.5	94.4	62.7	87.3	108.7	125.9
	0.65	28.9	40.3	50.2	58.1	43.4	60.4	75.3	87.1	57.8	80.6	100.4	116.2

Note: Assumptions include the following: 5% trucks, 0% buses, 0% RVs, 0.95 *PHF*, 3 ramps/mi, f_p = 1.00, 12-ft lanes, and 6-ft lateral clearance. Values do not represent specific segment characteristics.

Exhibit 10-9
Generalized Daily Service Volumes for Rural Freeway Facilities (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Freeways LOS B	LOS C	LOS D	LOS E	Six-Lane Freeways LOS B	LOS C	LOS D	LOS E	Eight-Lane Freeways LOS B	LOS C	LOS D	LOS E
Level Terrain													
0.09	0.50	41.1	54.9	66.2	75.3	61.6	82.3	99.3	112.9	82.2	109.8	132.4	150.5
	0.55	37.4	49.9	60.2	68.4	56.0	74.8	90.2	102.6	74.7	99.8	120.3	136.9
	0.60	34.2	45.7	55.1	62.7	51.4	68.6	82.7	94.1	68.5	91.5	110.3	125.5
	0.65	31.6	42.2	50.9	57.9	47.4	63.3	76.4	86.9	63.2	84.4	101.8	115.8
0.10	0.50	37.0	49.4	59.6	67.7	55.5	74.1	89.3	101.6	74.0	98.8	119.1	135.5
	0.55	33.6	44.9	54.1	61.6	50.4	67.4	81.2	92.4	67.2	89.8	108.3	123.2
	0.60	30.8	41.2	49.6	56.5	46.2	61.7	74.4	84.7	61.6	82.3	99.3	112.9
	0.65	28.4	38.0	45.8	52.1	42.7	57.0	68.7	78.2	56.9	76.0	91.6	104.2
0.11	0.50	33.6	44.9	54.1	61.6	50.4	67.4	81.2	92.4	67.2	89.8	108.3	123.2
	0.55	30.6	40.8	49.2	56.0	45.8	61.2	73.8	84.0	61.1	81.6	98.4	112.0
	0.60	28.0	37.4	45.1	51.3	42.0	56.1	67.7	77.0	56.0	74.8	90.2	102.6
	0.65	25.9	34.5	41.6	47.4	38.8	51.8	62.5	71.1	51.7	69.1	83.3	94.7
0.12	0.50	30.8	41.2	49.6	56.5	46.2	61.7	74.4	84.7	61.6	82.3	99.3	112.9
	0.55	28.0	37.4	45.1	51.3	42.0	56.1	67.7	77.0	56.0	74.8	90.2	102.6
	0.60	25.7	34.3	41.4	47.0	38.5	51.5	62.0	70.6	51.4	68.6	82.7	94.1
	0.65	23.7	31.7	38.2	43.4	35.6	47.5	57.3	65.1	47.4	63.3	76.4	86.9
Rolling Terrain													
0.09	0.50	36.9	49.3	59.4	67.6	55.4	74.0	89.2	101.4	73.8	98.6	118.9	135.2
	0.55	33.6	44.8	54.0	61.5	50.3	67.2	81.1	92.2	67.1	89.6	108.1	122.9
	0.60	30.8	41.1	49.5	56.3	46.1	61.6	74.3	84.5	61.5	82.2	99.1	112.7
	0.65	28.4	37.9	45.7	52.0	42.6	56.9	68.6	78.0	56.8	75.9	91.5	104.0
0.10	0.50	33.2	44.4	53.5	60.9	49.8	66.6	80.3	91.3	66.4	88.7	107.0	121.7
	0.55	30.2	40.3	48.6	55.3	45.3	60.5	73.0	83.0	60.4	80.7	97.3	110.6
	0.60	27.7	37.0	44.6	50.7	41.5	55.5	66.9	76.1	55.4	74.0	89.2	101.4
	0.65	25.6	34.1	41.2	46.8	38.3	51.2	61.7	70.2	51.1	68.3	82.3	93.6
0.11	0.50	30.2	40.3	48.6	55.3	45.3	60.5	73.0	83.0	60.4	80.7	97.3	110.6
	0.55	27.5	36.7	44.2	50.3	41.2	55.0	66.3	75.4	54.9	73.3	88.4	100.6
	0.60	25.2	33.6	40.5	46.1	37.7	50.4	60.8	69.2	50.3	67.2	81.1	92.2
	0.65	23.2	31.0	37.4	42.6	34.8	46.5	56.1	63.8	46.5	62.1	74.8	85.1
0.12	0.50	27.7	37.0	44.6	50.7	41.5	55.5	66.9	76.1	55.4	74.0	89.2	101.4
	0.55	25.2	33.6	40.5	46.1	37.7	50.4	60.8	69.2	50.3	67.2	81.1	92.2
	0.60	23.1	30.8	37.2	42.3	34.6	46.2	55.7	63.4	46.1	61.6	74.3	84.5
	0.65	21.3	28.4	34.3	39.0	31.9	42.7	51.4	58.5	42.6	56.9	68.6	78.0

Note: Assumptions include the following: 12% trucks, 0% buses, 0% RVs, 0.88 *PHF*, 0.2 ramp/mi, f_p = 0.85, 12-ft lanes, and 6-ft lateral clearance. Values do not represent specific segment characteristics.

ACTIVE TRAFFIC MANAGEMENT AND OTHER MEASURES TO IMPROVE PERFORMANCE

Active traffic management (ATM) consists of the dynamic and continuous monitoring and control of traffic operations on a facility to improve its performance. Examples of ATM measures include congestion pricing, ramp metering, changeable message signs, incident response programs, and speed harmonization (variable speed limits).

ATM measures can influence both the nature of demand for the facility and the ability of the facility to deliver the capacity tailored to serve the demand. ATM measures can improve facility performance, sometimes significantly.

Other advanced design and management measures, not specifically included in the definition of ATM, can also significantly improve facility performance. These measures include auxiliary lanes, narrow lanes, high-occupancy vehicle (HOV) lanes, temporary use of shoulders, and designated truck lanes and ramps.

This methodology does not reflect all these measures. However, ramp metering can be taken into account by altering on-ramp demands in accordance with metering rates. Auxiliary lanes and narrow lanes are taken into account in the segment methodologies for basic freeway segments and weaving segments.

Other measures are not accounted for in this methodology. Chapter 35 provides a more detailed discussion of ATM and other advanced design and management strategies and insight into how their impacts may be evaluated.

2. METHODOLOGY

The methodology presented in this chapter provides for the integrated analysis of a freeway facility composed of connected segments. The methodology builds on the models and procedures for individual segments, as described in Chapter 11, Basic Freeway Segments; Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments.

SCOPE OF THE METHODOLOGY

Because the freeway facility methodology builds on the segment methodologies of Chapters 11, 12, and 13, it incorporates all aspects of those chapters' methodologies. This methodology adds the ability to consider a number of linked segments over a number of time periods and to determine some overall operational parameters that allow for the assessment of a facility LOS and capacity.

This methodology also adds the ability to analyze operations when LOS F exists on one or more segments of the defined facility. In Chapters 11, 12, and 13, the existence of a breakdown (LOS F) is identified for a given segment, as appropriate. The segment methodologies do not, however, 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, there is a queue extending upstream that is not included in the facility definition and that therefore cannot be analyzed. When the last segment operates at LOS F, there may be a downstream bottleneck outside the facility definition. Again, the impacts of this congestion cannot be evaluated when it is not fully contained within the defined facility. LOS F in either the first or last time period creates similar problems with regard to time. If the first time period is at LOS F, then LOS F may exist in previous time periods as well. If the last time period is at LOS F, subsequent periods may be at LOS F as well. The impacts of a breakdown cannot be fully analyzed unless it is fully contained within the defined facility and defined total analysis period. The same problems would exist if the analysis were conducted by using simulation.

There is no limit to the number of time periods that can be analyzed. 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.

This methodology is based on research sponsored by the Federal Highway Administration (*1*).

LIMITATIONS OF THE METHODOLOGY

The methodology has the following limitations:

1. The methodology does not account for the 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 methodology can address local oversaturated flow but cannot directly address systemwide oversaturation flow conditions.
5. The completeness of the analysis will be limited if freeway segments in the first time interval, the last time interval, and the first freeway segment (in all time periods) have demand-to-capacity ratios greater than 1.00. The rationale for these limitations is discussed in the section on demand-to-capacity ratio.
6. The existence of HOV lanes on freeways raises the issues of the operating characteristics of such lanes and their effect on operating characteristics on the remainder of the freeway. The methodology does not directly address separated HOV facilities and does not account for the interactions between HOV lanes and mixed-flow lanes and the weaving that may be produced.
7. 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.
8. The method does not address toll plaza operations or their effect on freeway facility operations.

Given enough time, the analyst can analyze a completely undersaturated time–space domain manually, although it is very difficult and time-consuming. It is not expected that analysts will ever manually analyze a time–space domain that includes oversaturation. FREEVAL-2010 is a computational engine that can be used to implement the methodology, regardless of whether the time–space domain contains oversaturated segments and time periods. It is available in the Technical Reference Library section of Volume 4 of the *Highway Capacity Manual* (HCM).

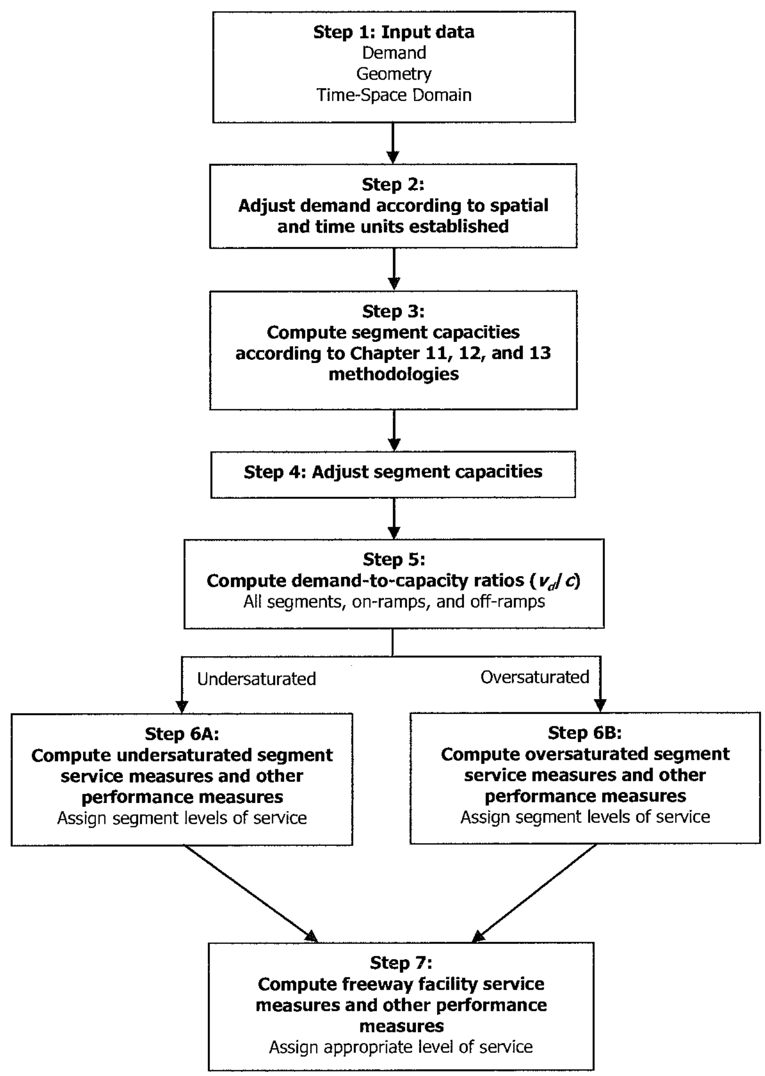
Because this chapter's methodology incorporates the methodologies for basic, weaving, merging, and diverging freeway segments, the limitations of those procedures also apply here.

The method does not include analysis of the street-side terminals of freeway on- and off-ramps. The methodologies of Chapters 18, 19, 20, and 21 should be used for intersections that are signalized, two-way STOP-controlled, all-way STOP-controlled, and roundabouts, respectively. Chapter 22, Interchange Ramp Terminals, provides a more comprehensive analysis of freeway interchanges where the street-side ramp terminals are signalized intersections or roundabouts.

OVERVIEW

Exhibit 10-10 summarizes the methodology for analyzing freeway facilities. The methodology adjusts vehicle speeds appropriately to account for the effects in adjacent segments. The methodology can analyze freeway traffic management strategies only in cases for which 15-min intervals are appropriate and for which reliable data for estimated capacity and demand exist.

Exhibit 10-10
Freeway Facility Methodology



COMPUTATIONAL STEPS

The purpose of this section is to describe 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 1: Input Data

Data concerning demand, geometry, and the time–space domain must be specified. As the methodology builds on segment analysis, all data for each segment and each time period must be provided, as indicated in Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments.

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 trucks (%),
- Percent RVs (%), and
- Driver population factor (f_p).

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 not recommended, however, as 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).

The driver population factor is normally 1.00, unless the driver population is dominated by unfamiliar users, in which case a value between 0.85 and 1.00 is assigned, on the basis of local characteristics and knowledge.

Geometry

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

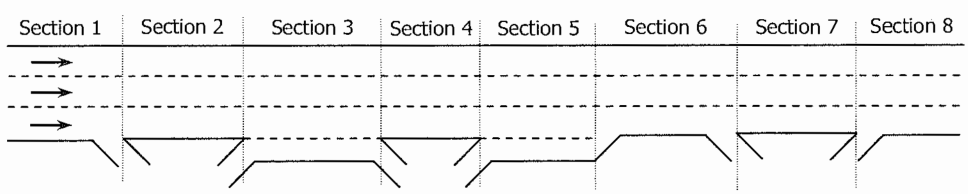
- Number of lanes;
- Average lane width;
- Right-side lateral clearance;
- Terrain;
- Free-flow speed; and
- Location of merge, diverge, and weaving segments, with all internal geometry specified, including the number of lanes on ramps and at ramp–freeway junctions or within weaving segments, lane widths, 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.

Time–Space Domain

A time–space domain for the analysis must be established. The domain consists of a specification of the freeway *sections* included in the defined facility and an identification of the time intervals for which the analysis is to be conducted. A typical time–space domain is shown in Exhibit 10-11.

Exhibit 10-11
Example Time–Space Domain for Freeway Facility Analysis



Time Step	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	Section 7	Section 8
1								
2								
3								
4								
5								
6								
7								
8								

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 or where a lane is added or dropped. 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 12 for weaving segments and in Chapter 13 for merge and diverge segments.

The vertical scale indicates the study time duration. Time extends down the time–space domain, and the scale is divided into 15-min intervals. In the example shown, there are 8 sections and 8 time steps, yielding $8 \times 8 = 64$ time–space cells, each of which will be analyzed within the methodology.

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 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.

Converting the Horizontal Scale from Sections to Analysis Segments

The sections of the defined freeway facility are established by using points where demand changes or where lanes are added or subtracted. This, however, does *not* fully describe individual *segments* for analysis within the methodology. The conversion from sections to analysis segments can be done manually by applying the principles discussed here.

Chapter 13, 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.

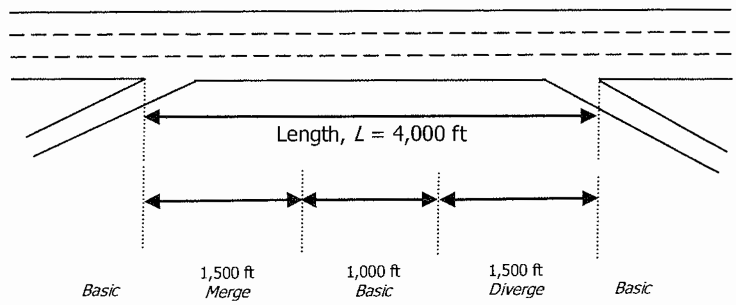
Consider the illustration of Exhibit 10-12. It shows a one-lane on-ramp followed by a one-lane off-ramp with no auxiliary lane between them. The illustration assumes that there are no upstream or downstream ramps or weaving segments that impinge on this section.

In Exhibit 10-12(a), there are 4,000 ft between the two ramps. Therefore, the merge segment extends 1,500 ft downstream, and the diverge segment extends 1,500 ft upstream, which leaves a 1,000-ft basic freeway segment between them.

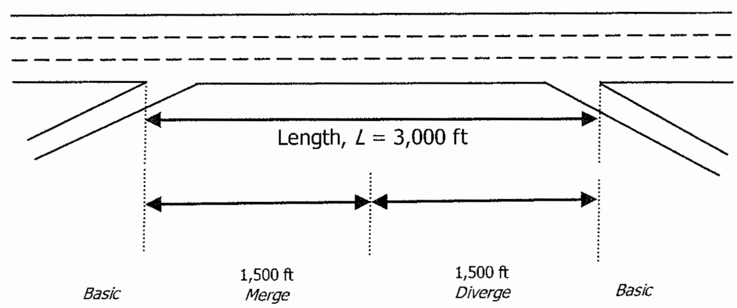
In Exhibit 10-12(b), there are 3,000 ft between the two ramps. The two 1,500-ft ramp influence areas define the entire length. Therefore, there is no basic freeway segment between the merge and diverge segments.

In Exhibit 10-12(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.

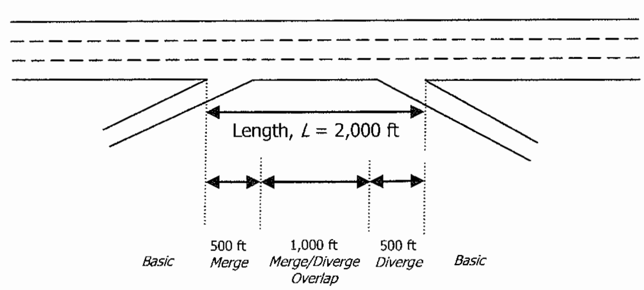
Exhibit 10-12
Defining Analysis Segments for a Ramp Configuration



(a) Length between ramps = 4,000 ft



(b) Length between ramps = 3,000 ft



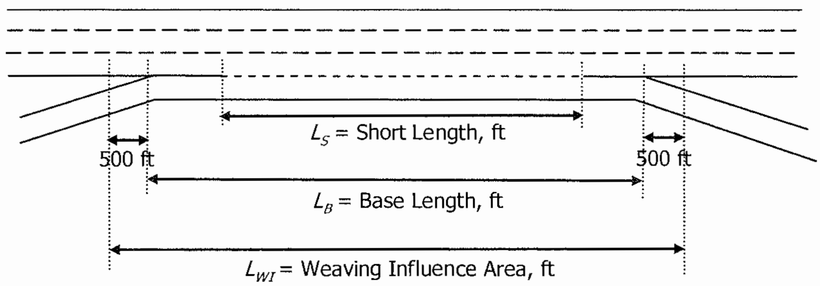
(c) Length between ramps = 2,000 ft

Chapter 13, Freeway Merge and Diverge Segments, covers this situation. Where ramp influence areas overlap, the analysis is conducted for each ramp separately. The analysis producing the worst LOS (or service measure value if the LOS is equivalent) is used to define operations in the overlap area.

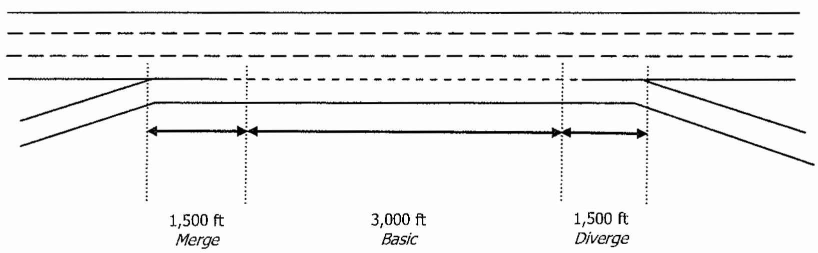
The facility methodology goes through the logic of distances and segment definitions to convert section boundaries to segment boundaries for analysis. If the distance between an on-ramp and off-ramp is less than the full influence area of 1,500 ft, the worst case is applied to the distance between the 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 10-13 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}, then the entire segment is analyzed as a weaving segment, as shown in Exhibit 10-13(a).

Exhibit 10-13
Defining Analysis Segments for a Weaving Configuration



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



(b) Case II: $L_B > L_{wMAX}$ (isolated merge and diverge exists)

Three lengths are involved in analyzing a weaving segment:

- 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);
- The influence area of the weaving segment (L_{WI}), which includes 500 ft upstream and downstream of L_B; and
- The short length of the segment, defined as the distance over which lane changing is not prohibited or dissuaded by markings (L_S).

The latter is the length that is used in all the predictive models for weaving segment analysis. The results of these models, however, apply to a distance of L_B + 500 ft upstream and L_B + 500 ft downstream. For further discussion of the various lengths applied to weaving segments, consult Chapter 12.

If the distance between the merge and diverge points is greater than L_{wMAX}, then the merge and diverge segments are too far apart to form a weaving segment. As shown in Exhibit 10-13(b), the merge and diverge segments are treated separately, and any distance remaining between the merge and diverge influence areas is treated as a basic freeway segment.

In the Chapter 12 weaving methodology, the value of L_{wMAX} depends on a number of factors, 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 separate merge, diverge, and possibly basic segments in others.

In segmenting the freeway facility for analysis, merge, diverge, and weaving segments are identified as illustrated in Exhibit 10-12 and Exhibit 10-13. 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, then the level terrain portion and the specific grade portion would be established as two separate, consecutive basic freeway segments.

Step 2: Adjust Demand According to Spatial and Time Units Established

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 that there is not a 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 input 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 every time interval can be estimated. The segment demands can be thought of as filtering across the time–space domain and filling each cell of the time–space matrix.

Demand estimation is needed if the methodology uses 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. 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 10-6 and Equation 10-7 summarize this process.

$$f_{TISi}=\frac{\sum_{j} V_{ON15ij}}{\sum_{j} V_{OFF15ij}}$$

Equation 10-6

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

Equation 10-7

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 10-11, with each cell containing an estimate of its 15-min demand.

Because each time period is separately balanced, it is advisable to limit the total length of the defined facility to a distance that can be traversed within 15 min. In practical terms, this practice limits the length of the facility to 9 to 12 mi.

Step 3: Compute Segment Capacities According to Chapter 11, 12, and 13 Methodologies

Segment capacity estimates are determined by the methodologies of Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 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 known bottlenecks exist.

On-ramp and off-ramp roadway capacities are also determined in this step with the Chapter 13 methodology. On-ramp demands may exceed on-ramp capacities and limit the traffic demand entering the facility. Off-ramp demands may exceed off-ramp capacities and 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 feature not only allows for evaluating a prescribed ramp-metering plan but also permits the user to improve the ramp-metering plan through experimentation.

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 4: Adjust Segment Capacities

Segment capacities can be affected by a number of conditions not normally accounted for in the segment methodologies of Chapters 11, 12, and 13. These reductions include the effects of short-term and long-term lane closures for construction or major maintenance operations, the effects of adverse weather conditions, and the effects of other environmental factors.

At lane drops, permanent reductions in capacity occur. They are included in the base methodology, which automatically accounts for the capacity of segments on the basis of the number of lanes in the segment and other prevailing conditions.

Capacity Reductions due to Construction and Major Maintenance Operations

Capacity reductions due to construction activities can be divided into short-term work-zone lane closures, typically for maintenance, and long-term lane closures, typically for construction. A primary distinction between short-term work zones and long-term construction zones is the nature of the barriers used to demarcate the work area. Long-term construction zones generally use portable concrete barriers, while short-term work zones use standard channeling devices (e.g., traffic cones, drums) in accordance with the *Manual on Uniform Traffic Control Devices for Streets and Highways* (*2*). Capacity reductions due to long-term construction or major maintenance operations generally last several weeks, months, or even years, depending on the nature of the work. Short-term closures generally last a few hours.

Short-Term Work Zones

Research (*3*) suggests that a capacity of 1,600 pc/h/ln be used for short-term freeway work zones, regardless of the lane-closure configuration. However, for some types of closures, a higher value could be appropriate.

This base value should be adjusted for other conditions, as follows:

1. *Intensity of work activity:* The intensity of work activity refers to the number of workers on the site, the number and size of work vehicles in use, and the proximity of the work activity to the travel lanes. Unusual types of work also contribute to intensity in terms of rubbernecking by drivers passing through the site. Research (*3*) suggests that the base value of 1,600 pc/h/ln be adjusted by as much as ±10% for work activity that is more or less intensive than normal. It does not, however, define what constitutes "normal" intensity, so this factor should be applied on the basis of professional judgment and local experience.

6. *Effects of heavy vehicles:* Because the base value is given in terms of pc/h/ln, it is recommended that the heavy vehicle adjustment factor (f_{HV}) be applied. A complete discussion of the heavy vehicle adjustment factor and its determination are included in Chapter 11, Basic Freeway Segments. Equation 10-8 shows how the factor is determined.

$$f_{HV} = \frac{1}{1+P_T(E_T-1)+P_R(E_R-1)}$$

Equation 10-8

where

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of trucks and buses in the traffic stream,

P_R = proportion of RVs in the traffic stream,

E_T = passenger-car equivalent for trucks and buses, and

E_R = passenger-car equivalent for RVs.

Passenger-car equivalents for trucks and buses and for RVs may be found in Chapter 11, Basic Freeway Segments.

7. *Presence of ramps:* If there is an entrance ramp within the taper area approaching the lane closure or within 500 ft downstream of the beginning of the full lane closure, the ramp will have a noticeable effect on the capacity of the work zone for handling mainline traffic. This situation arises in two ways: (*a*) the ramp traffic generally forces its way in, so it directly reduces the amount of mainline traffic that can be handled, and (*b*) the added turbulence in the merge area may slightly reduce capacity (even though such turbulence does *not* reduce capacity on a normal freeway segment without lane closures). If at all possible, on-ramps should be located at least 1,500 ft upstream of the beginning of the full lane closure to maximize the total work zone throughput. If that cannot be done, then either the ramp volume should be added to the mainline volume to be served or the capacity of the work zone should be decreased by the ramp volume (up to a maximum of one-half of the capacity of one lane) on the assumption that, at very high volumes, mainline and ramp vehicles will alternate.

Equation 10-9 is used to estimate the resulting reduced capacity in vehicles per hour.

$$c_a = \{[(1{,}600+I)\times f_{HV}]\times N\}-R$$

Equation 10-9

where

c_a = adjusted mainline capacity (veh/h);

I = adjustment factor for type, intensity, and proximity of work activity, pc/h/ln (ranges between ±160 pc/h/ln);

f_{HV} = heavy-vehicle adjustment factor;

N = number of lanes open through the work zone; and

R = manual adjustment for on-ramps (veh/h).

Long-Term Construction Zones

There have been many studies of long-term construction zone capacities. They are summarized in Exhibit 10-14.

Exhibit 10-14
Capacity of Long-Term Construction Zones (veh/h/ln)

State	Normal Lanes to Reduced Lanes						Source
	2 to 1	**3 to 2**	**3 to 1**	**4 to 3**	**4 to 2**	**4 to 1**	
TX	1,340		1,170				(*4*)
NC	1,690		1,640				(*5*)
CT	1,500–1,800		1,500–1,800				(*6*)
MO	1,240	1,430	960	1,480	1,420		(*7*)
NV	1,375–1,400		1,375–1,400				(*6*)
OR	1,400–1,600		1,400–1,600				(*6*)
SC	950		950				(*6*)
WA	1,350		1,450				(*6*)
WI	1,560–1,900		1,600–2,000		1,800–2,100		(*6, 8*)
FL	1,800		1,800				(*9*)
VA	1,300	1,300	1,300	1,300	1,300	1,300	(*10*)
IA	1,400–1,600	1,400–1,600	1,400–1,600	1,400–1,600	1,400–1,600	1,400–1,600	(*11*)
MA	1,340	1,490	1,170	1,520	1,480	1,170	(*12*)
Default	**1,400**	**1,450**	**1,450**	**1,500**	**1,450**	**1,350**	

Source: Adapted from Chatterjee et al. (*13*).

It is easy to see from Exhibit 10-14 that capacities through long-term construction zones are highly variable and depend on many site-specific characteristics. Therefore, it is better to base this adjustment on local data and experience. If such data do not exist and cannot be reasonably acquired, the default values of Exhibit 10-14 may be used to provide an approximate estimate of construction zone capacity.

Lane-Width Consideration

The impact of lane width on general freeway operations is incorporated into the methodology of Chapter 11, Basic Freeway Segments, for determining free-flow speed. As free-flow speed affects capacity, it follows that restricted lane widths will negatively affect capacity.

As free-flow speeds are not estimated specifically for work or construction zones, it is appropriate to add an adjustment factor for the effect of lane widths narrower than 12 ft in a work or construction zone. The factor f_{LW} would be added to Equation 10-9, as shown in Equation 10-10:

Equation 10-10

$$c'_a = c_a \times f_{LW}$$

where c'_a is the adjusted capacity of the work or construction zone reflecting the impact of restricted lane width, in vehicles per hour, and all other variables are as previously defined.

The value of the adjustment factor f_{LW} is 1.00 for 12-ft lanes, 0.91 for lanes between 10.0 and 11.9 ft, and 0.86 for lanes between 9.0 and 9.9 ft. If lanes narrower than 9.0 ft are in use, local observations should be made to calibrate an appropriate adjustment.

Capacity Reductions due to Weather and Environmental Conditions

A number of studies have attempted to address the impacts of adverse weather and environmental conditions on the capacity of freeways. Comprehensive results for a range of conditions in Iowa, summarized in Exhibit 10-15, are provided elsewhere (*14*).

Exhibit 10-15
Capacity Reductions due to Weather and Environmental Conditions in Iowa

Type of Condition	Intensity of Condition	Percent Reduction in Capacity	
		Average	Range
Rain	>0 ≤ 0.10 in./h	2.01	1.17–3.43
	>0.10 ≤ 0.25 in./h	7.24	5.67–10.10
	>0.25 in./h	14.13	10.72–17.67
Snow	>0 ≤ 0.05 in./h	4.29	3.44–5.51
	>0.05 ≤ 0.10 in./h	8.66	5.48–11.53
	>0.10 ≤ 0.50 in./h	11.04	7.45–13.35
	>0.50 in./h	22.43	19.53–27.82
Temperature	<50°F ≥ 34°F	1.07	1.06–1.08
	<34°F ≥ –4°F	1.50	1.48–1.52
	<–4°F	8.45	6.62–10.27
Wind	>10 ≤ 20 mi/h	1.07	0.73–1.41
	>20 mi/h	1.47	0.74–2.19
Visibility	<1 ≥ 0.50 mi	9.67	One site
	<0.50 ≤ 0.25 mi	11.67	One site
	<0.25 mi	10.49	One site

Source: Adapted from Agarwal et al. (*14*).

Additional information is available in the literature. Additional data and information on the impacts of rain on freeway capacity are provided elsewhere (*15, 16*), as are information on the effects of snow (*16*) and insights and information on the effects of fog (*17, 18*).

A study of capacity on German autobahns provides data on the difference between daytime and nighttime conditions on wet or dry pavements (*19*). Exhibit 10-16 summarizes these results.

Exhibit 10-16
Capacities on German Autobahns Under Various Conditions (veh/h/ln)

Freeway Lanes	Weekday or Weekend	Daylight Dry	Dark Dry	Daylight Wet	Dark Wet
6	Weekday (% change*)	1,489	1,299 (13%)	1,310 (12%)	923 (38%)
6	Weekend (% change*)	1,380	1,084 (21%)	1,014 (27%)	— —
4	Weekday (% change*)	1,739	1,415 (19%)	1,421 (18%)	913 (47%)
4	Weekend (% change*)	1,551	1,158 (25%)	1,104 (29%)	— —

Note: *Percent change from daylight, dry conditions for the same day of week.
Source: Adapted from Brilon and Ponzlet (*19*).

This exhibit is interesting in that the daylight, dry capacities of German autobahns are somewhat less than might be expected on U.S. freeways. This situation could be due to the higher speeds that prevail on the autobahns and heavy-vehicle presence, which are not reflected in these veh/h/ln statistics.

The daylight wet versus dry capacity reductions are greater in Exhibit 10-16 than those shown in Exhibit 10-15, which may again be a reflection of different driver behavior characteristics in Germany and the United States. Darkness alone has a significant impact on autobahn capacities. Since winter peak hours occur when it is dark in many areas of the country, such reductions are important to recognize.

The difference between weekday and weekend capacities is also interesting and is on the order of 7% to 10% in Exhibit 10-16. This impact is generally reflected in the use of a driver-population factor f_p (see Chapter 11). Weekend driving populations may not be as familiar with the facility as weekday commuters. Even familiar users may not drive as aggressively on weekend recreational or other trips when the pressure of a specific schedule may be less than is present during the week.

Capacity Reductions due to Traffic Accidents or Vehicular Breakdowns

Capacity reductions due to traffic accidents or other incidents are generally short-lived, ranging from less than 1 h before they can be cleared to as long as 12 h for an accident involving severe injuries, fatalities, hazardous materials cleanup, or cleanup of other materials from vehicles involved in accidents.

One study (*20*) reported the mean duration of a traffic incident to be 37 min, with more than half the incidents lasting 30 min or less and 82% lasting less than 1 h.

Exhibit 10-17 summarizes the results of two studies (*21, 22*) on the capacity impacts of lane blockages due to incidents, including accidents. An incident's effect on capacity depends on the proportion of the traveled roadway that is blocked and on the number of lanes on the freeway at that point.

Exhibit 10-17
Proportion of Freeway Segment Capacity Available Under Incident Conditions

Number of Lanes (One Direction)	Shoulder Disablement	Shoulder Accident	One Lane Blocked	Two Lanes Blocked	Three Lanes Blocked
2	0.95	0.81	0.35	0.00	N/A
3	0.99	0.83	0.49	0.17	0.00
4	0.99	0.85	0.58	0.25	0.13
5	0.99	0.87	0.65	0.40	0.20
6	0.99	0.89	0.71	0.50	0.26
7	0.99	0.91	0.75	0.57	0.36
8	0.99	0.93	0.78	0.63	0.41

In a blocked lane, the loss of capacity is likely to be greater than the proportion of the roadway that is blocked. A one-lane blockage on a two-lane directional freeway segment (50% of the roadway blocked) reduces capacity to 35% of the original value, for example. The added loss of capacity arises because drivers slow to look at the incident while they are abreast of it and are slow to react to the possibility of speeding up to move through the incident area.

The "rubbernecking" factor is also responsible for a reduction in capacity in the direction of travel opposite to that in which the accident or incident occurred. While no quantitative studies of this impact have been conducted, experience suggests that the severity of the accident or incident plays a significant role in the impact of rubbernecking. The reduction in capacity may range from 5% for a single-vehicle accident with one emergency vehicle present to as high as 25% for a multivehicle accident with several emergency vehicles.

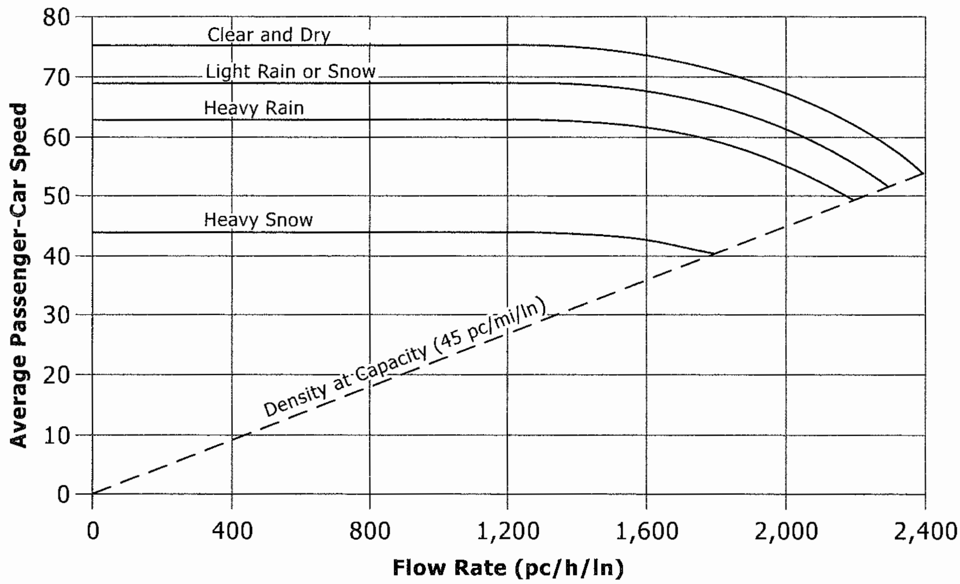
Applying Capacity Reductions

There are several ways to use the information on reduced capacities discussed in this section.

Quick approximations simply require that the capacity of each freeway facility segment (as estimated by using the methodologies of Chapters 11, 12, and 13) be reduced by all the impacts of work zones, weather, environment, and accidents or incidents that are present, in accord with the information provided here. The methodology continues using these reduced capacities.

If speed information is available, then the free-flow speed through the restricted capacity area can be used to select an appropriate speed–flow curve for analysis (from Chapter 11). The reduced free-flow speed results in a reduced capacity. An example of this approach is illustrated in Exhibit 10-18, which is based on speed data presented elsewhere (*16, 19*).

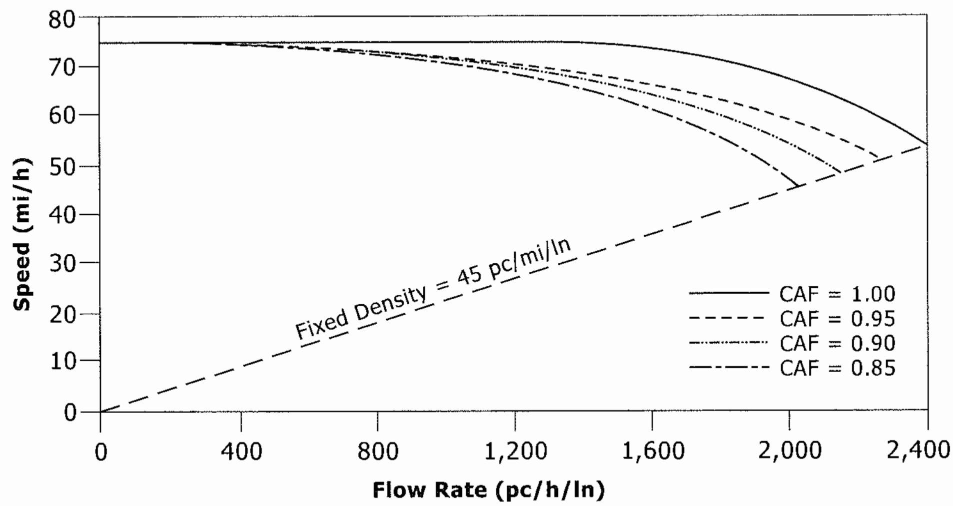
Exhibit 10-18
Illustration of Speed–Flow Curves for Different Weather Conditions



Note: Free-flow speed = 75 mi/h (base conditions).

For most temporary capacity reductions, the only information available relates to capacity. In most of these cases, speed conditions can be reasonably estimated. For example, in construction zones, a reduced speed limit is usually posted, and lower speeds can be expected to occur, particularly when actual construction operations are taking place. Likewise, for incidents, traffic naturally slows as drivers pass the incident site, where rubbernecking takes place. Exhibit 10-19 shows an example of modeling such cases on the basis of a downward-shifted speed–flow curve.

Exhibit 10-19
Illustration of Adjusted Speed–Flow Curves for Indicated Capacity Reductions



Note: Free-flow speed = 75 mi/h (base conditions); CAF = capacity adjustment factor (proportion of available capacity).

If the analyst has no interest in speeds, the capacity reduction could be modeled by using a fractional number of lanes that would reflect the new capacity of the roadway rather than the actual number of lanes present. For example, in the case of a four-lane directional freeway segment with two lanes blocked, Exhibit 10-17 indicates that only 25% of capacity would be available. This segment could be modeled as if only one lane were available through the incident (even though two are actually in use).

Some of the performance measures that result from this methodology, however, rely on speed. A simple approach that does not deal with speed consequences would result in an incomplete analysis. Consequently, an approach that uses modified speed–flow curves, as illustrated here, is recommended.

Step 5: 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, then 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, then 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, then 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, then 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 6A. If Case 2 exists, the analysis moves to Step 6B.

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 18–22, 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 6A and 6B: Compute Undersaturated (6A)/Oversaturated (6B) Service Measures and Other 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.

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 it cannot handle a flow greater than its capacity, the following impacts will occur:

1. The v_d/c ratio of the bottleneck cell will be exactly 1.00, as 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 gets through. Downstream cells are subject to demand starvation due to the bottleneck.
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. This 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 Chapter 11, Basic Freeway Segments; Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments. Facility service and performance measures may then be estimated.

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 11, 12, and 13 will result in estimating a density D and a space mean speed S.

When the analysis moves from isolated segments to a system, 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.

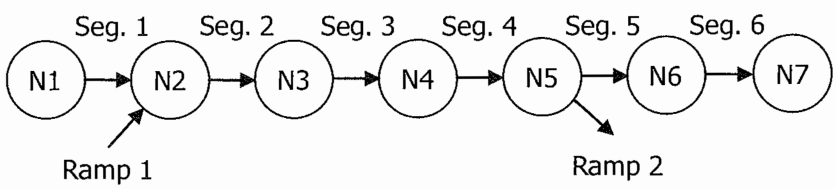
For each time interval, Equation 10-2 is used to estimate the average density for the defined freeway facility. This result is compared with the criteria of Exhibit 10-7 to determine the facility LOS for the time period. Each time period will have a separate LOS. Although LOS is not averaged over time intervals, if desired, density can be averaged over time intervals.

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.

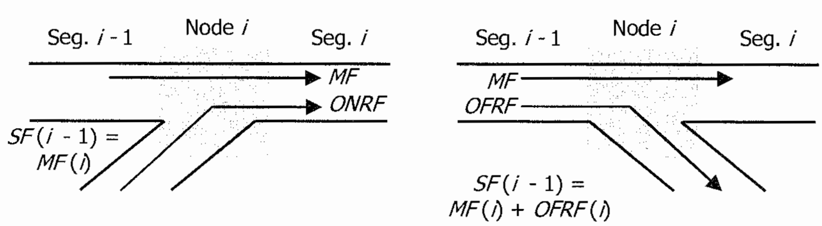
Exhibit 10-20 illustrates the node–segment concept. A node is defined as the junction of two segments. Given that there is a node at the beginning and end of the freeway facility, there will always be one more node than the number of segments on the facility.

Exhibit 10-20
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 10-21.

Exhibit 10-21
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 past performance of an individual variable. 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, it is necessary to begin the process 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 each segment, the expected demand is calculated from the demands for and capacities of the segment, including 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 11, 12, and 13.

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 is illustrated in Exhibit 10-21.

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. The 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 very 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 based on 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, then 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 10-20. The number of vehicles on each segment in the current time step is calculated with the following information:

- 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, then the performance measures are calculated from Chapters 11, 12, and 13 as appropriate.

If there was a queue on the current segment during the time interval, then 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 7: Compute Freeway Facility Service Measures and Other Performance Measures by Time Interval

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

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 (with Equation 10-2) and used to compare with the criteria of Exhibit 10-7.

3. APPLICATIONS

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

This chapter's methodology is sufficiently complex to require software for its application. Even for fully undersaturated analyses, the number and complexity of computations make it difficult and extremely time-consuming to analyze a case manually. Oversaturated analyses are considerably more complex, and manual solutions would be impractical. The computational engine for this methodology is FREEVAL-2010. A complete user's guide and executable spreadsheet are available in the Technical Reference Library in Volume 4.

OPERATIONAL ANALYSIS

The only mode in which the methodology can be directly implemented is operational analysis—that is, given a complete description of a freeway facility, its component segment geometries, and all relevant demand flow rates, a complex analysis is conducted of each segment, and of the freeway facility, by time interval. Outputs will include segment flow rates, densities, and average speeds as well as average facility density and speed for each time interval. By using the estimated facility density for each time interval, a facility LOS can be assigned.

Exhibit 10-22 shows the data inputs that are required for an operational analysis of a freeway facility.

Exhibit 10-22
Required Input Data for Freeway Facility Analysis

Geometric Data for Each Section
• Section length (ft) • Mainline number of lanes • Mainline average lane width (ft) • Mainline lateral clearance (ft) • Terrain (level, rolling, or mountainous), or specific grade (% grade, length in mi) • Ramp number of lanes • Ramp acceleration or deceleration lane length (ft) • Existence of independent HOV lane
Traffic Characteristic Data for Each Segment
• Mainline free-flow speed (mi/h), optional • Vehicle occupancy (passengers/veh) • Percent trucks and buses (%) • Percent RVs (%) • Driver population (commuter or recreational) • Ramp free-flow speeds (mi/h)
Demand Data for Each Segment
• Mainline entry demand for each time interval (veh/h) • On-ramp demands for each time interval (veh/h) • Off-ramp demands for each time interval (veh/h) • Weaving demand on weaving segments, by movement (veh/h) • HOV lane demand (veh/h), if present

Where all data are not readily available or collectable, the analysis may be supplemented by using consistent default values for each segment. Lists and discussions of default values are found in Chapter 11, Basic Freeway Segments;

Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments.

Performance measures output by the methodology for individual segments and the facility (for a given time interval) include the following:

- Average speed (mi/h),
- Average density (pc/mi/ln),
- Vehicle miles of travel,
- Vehicle hours of travel, and
- Travel time (min/veh).

Chapter 25 details facilitywide performance measure calculations by time interval.

PLANNING, PRELIMINARY ENGINEERING, AND DESIGN ANALYSIS

This methodology cannot be directly used in planning, preliminary engineering, and design applications. However, for generalized planning, Exhibit 10-8 (urban freeways) and Exhibit 10-9 (rural freeways) provide daily-service-volume tables for a variety of typical freeway conditions. 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.

Preliminary engineering and design applications of the methodology are possible by using the segment procedures described in Chapters 11, 12, and 13. Various geometric scenarios can be evaluated and compared by using a travel demand matrix and the facility methodology on the basis of the segment results.

TRAFFIC MANAGEMENT STRATEGIES

The freeway facilities methodology has incorporated procedures for assessing a variety of traffic management strategies. The methodology permits modifying previously calculated cell demands or capacities (or both) within the time–space domain to assess a traffic management strategy or a combination of strategies.

1. A growth factor parameter has been incorporated 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 evaluating a predetermined ramp-metering plan and experimenting 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 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.

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

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 by 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 10-23 provides a list of the limitations stated earlier in this chapter, along with their potential for improved treatment by alternative tools.

Exhibit 10-23
Limitations of the HCM Freeway Facilities Analysis Procedure

Limitation	Potential for Improved Treatment by 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 a variety of performance measures for individual segments along a freeway facility, and the entire facility, given each segment's traffic demand and characteristics. The following performance measures are reported by the freeway facilities procedure:

- 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, and operating costs. As with most other procedural chapters in the HCM, simulation outputs—especially graphics-based presentations—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 that are produced by the procedures in this chapter.

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

To better 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 undersaturated conditions—that is, v_d/c is less than 1.0 for all segments and time periods—the approach is generally disaggregate. In other words, the facility is subdivided into segments corresponding to basic freeway, weaving, and merge/diverge segments, and the LOS results are reported for individual segments on the basis of the analysis procedures of Chapters 11, 12, and 13, respectively. However, LOS results are not reported for the facility as a whole.

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 to 60 s, depending on the length of the shortest segment. 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. For oversaturated segments, the average segment density is calculated by dividing the average number of vehicles for all time steps (in the time interval) 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 11, 12, and 13.

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 both saturated and undersaturated conditions, tracking each vehicle through time and space and generally handling 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, while others may queue vehicles in the vertical, rather than 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 when 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, capacity is a function of the specified free-flow speed (which can be adjusted by lane width, shoulder width, and ramp density). In a simulation tool, capacity is typically a function of the specified minimum vehicle entry headway (into the system) and car-following parameters (assuming microscopic simulation).

While the determination of capacity for a basic freeway segment is clearly described in Chapter 11, this chapter does not offer specific guidance on determining the appropriate capacity for different segment types within a facility, other than to refer the reader to the individual chapters (basic segments, weaving segments, merge segments, diverge segments) for appropriate capacity values. The HCM specifies the capacity of a freeway facility in units of veh/h rather than pc/h.

In macroscopic simulation tools, capacity is generally an input. Thus, for this situation, it is straightforward to match the simulation capacity to the HCM capacity. Microscopic simulation tools, however, 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. Therefore, 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 model. Thus, given other factors and car-following 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 careful 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, this situation may lead to a difference between the HCM and simulation results. 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 13 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 trucks, buses, and RVs in the traffic stream as well as type of terrain (grade profile and its length). 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 into 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 passenger car equivalent values contained in the HCM were developed from simulation data, simplifying assumptions made to make them implementable in an analytical procedure result in some loss of fidelity in the treatment of different vehicle types.

Furthermore, it should be recognized that the 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., free-flow speed, gap acceptance threshold). However, it should also be recognized that the empirical data some HCM procedures are based on include the effects of the various driver types present in traffic streams.

Free-Flow Speed

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

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, then 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 that involve 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. 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.

4. EXAMPLE PROBLEMS

Exhibit 10-24
List of Example Problems

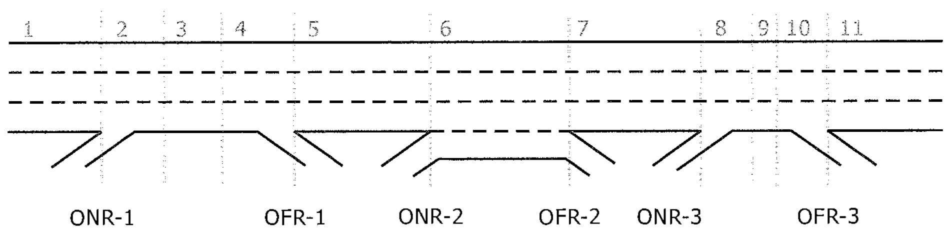
Example Problem	Description	Application
1	Evaluation of an undersaturated facility	Operational analysis
2	Evaluation of an oversaturated facility	Operational analysis
3	Capacity improvements to an oversaturated facility	Operational analysis

EXAMPLE PROBLEM 1: EVALUATION OF AN UNDERSATURATED FACILITY

The Facility

The subject of this operational analysis is an urban freeway facility 6 mi long and composed of 11 individual analysis segments, as shown in Exhibit 10-25.

Exhibit 10-25
Freeway Facility in Example Problem 1



The facility has three on-ramps and three off-ramps. Geometric details are given in Exhibit 10-26.

Exhibit 10-26
Geometry of Directional Freeway Facility for Example Problem 1

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	3	3	3	3	3

Note: B = basic freeway segment, W = weaving segment, ONR = on-ramp (merge) segment, OFR = off-ramp (diverge) segment, R = overlapping ramp segment.

The on- and off-ramps in Segment 6 are connected by an auxiliary lane and the segment may therefore operate as a weaving segment, depending on traffic patterns. The separation of the on-ramp in Segment 8 and the off-ramp in Segment 10 is less than 3,000 ft. Since the ramp influence area of on-ramps and off-ramps is 1,500 ft, according to Chapter 13, the segment affected by both ramps is analyzed as a separate overlapping ramp segment (Segment 9), labeled "R."

The analysis question at hand is the following: What is the operational performance and LOS of the directional freeway facility shown in Exhibit 10-25?

The Facts

In addition to the information contained in Exhibit 10-25 and Exhibit 10-26, the following characteristics of the freeway facility are known:

Heavy vehicles = 5% trucks, 0% RVs (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp FFS = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for FFS = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level; and

Analysis duration = 75 min (divided into five 15-min intervals).

Comments

The facility was segmented into analysis segments on the basis of the guidance given in this chapter. The facility shown in Exhibit 10-25 initially depicts seven freeway *sections* (measured between ramps) that are divided into 11 analysis *segments.* The facility contains each of the possible segment types for illustrative purposes, including basic segment (B), weaving segment (W), merge segment (ONR), diverge segment (OFR), and overlapping ramp segment (R). The input data contain the required information needed for each of the segment methodologies.

The classification of the weave in Segment 6 is preliminary until it is determined whether the segment operates as a weave. For this purpose, the short length must be compared with the maximum length for weaving analysis to determine whether the Chapter 12, Weaving Segments, methodology or the Chapter 11, Basic Freeway Segments, methodology is applicable. The short length of the weaving segment used for calculation is shorter than the weaving influence area over which the calculated speed and density measures are applied.

Chapter 11 must be consulted to find appropriate values for the heavy vehicle adjustment factor f_{HV} and the driver population adjustment factor f_p. FREEVAL-2010 automatically determines these adjustment factors for general terrain conditions, but user input is needed for specific upgrades and composite grades.

All input parameters have been specified, so default values are not needed. Fifteen-minute demand flow rates are given in vehicles per hour under prevailing conditions. These demands must be converted to passenger cars per hour under equivalent ideal conditions for use in the parts of the methodology related to segment LOS estimation.

Step 1: Input Data

Traffic demand inputs for all 11 segments and five analysis intervals are given in Exhibit 10-27.

Exhibit 10-27
Demand Inputs for Example Problem 1

Time Step (15 min)	Entering Flow Rate (veh/h)	Ramp Flow Rates by Time Period (veh/h)						Exiting Flow Rate (veh/h)
		ONR1	ONR2*	ONR3	OFR1	OFR2	OFR3	
1	4,505	450	540 (50)	450	270	360	270	5,045
2	4,955	540	720 (100)	540	360	360	270	5,765
3	5,225	630	810 (150)	630	270	360	450	6,215
4	4,685	360	360 (80)	450	270	360	270	4,955
5	3,785	180	270 (50)	270	270	180	180	3,875

* Numbers in parentheses indicate ONR-2 to OFR-2 demand flow rates in Weaving Segment 6.

The volumes in Exhibit 10-27 represent the 15-min demand flow rates on the facility as determined from field observations or other sources. The actual volume served in each segment will be determined by the methodology. The demand flows are given for the extended time–space domain, consistent with this chapter's recommendations. Peaking occurs in the third 15-min period. Since inputs are in the form of 15-min flow rates, no peak hour factor adjustment is necessary. Additional geometric and traffic-related inputs are as specified in Exhibit 10-25 and the facts section of the problem statement.

Step 2: Demand Adjustments

The traffic flows in Exhibit 10-27 are already given in the form of actual demands. Therefore, no additional demand adjustment is necessary, since the flows represent true demand. Demand adjustment is necessary only if field-measured volumes are used that may be affected by upstream congestion (bottleneck) on the facility. The methodology (and FREEVAL-2010) assume that the user inputs true demand flows.

Step 3: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments. The resulting capacities are shown in Exhibit 10-28. Since the capacity of a weaving segment depends on traffic patterns, including the weaving ratio, it varies by time period. The remaining segment capacities are constant in all five time intervals. The capacities for Segments 1–5 and 7–11 are the same, since the segments have the same basic cross section. The units shown are in vehicles per hour.

Exhibit 10-28
Segment Capacities for Example Problem 1

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1						8,252					
2						8,261					
3	6,732	6,732	6,732	6,732	6,732	8,303	6,732	6,732	6,732	6,732	6,732
4						8,382					
5						8,442					

Step 4: Adjust Segment Capacities

This step typically allows the user to adjust capacities of specific segments or time periods to model the effects of short-term work zones, long-term construction, inclement weather conditions, or incidents. Since it is the base scenario in this sequence of example problems, no additional capacity adjustments are performed.

Step 5: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios are calculated from the demand flows in Exhibit 10-27 and from the segment capacities in Exhibit 10-28.

Exhibit 10-29
Segment Demand-to-Capacity Ratios for Example Problem 1

Time Step	Demand-to-Capacity Ratios by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	0.67	0.74	0.74	0.74	0.70	0.63	0.72	0.79	0.79	0.79	0.75
2	0.74	0.82	0.82	0.82	0.76	0.71	0.82	0.90	0.90	0.90	0.86
3	0.78	0.87	0.87	0.87	0.83	0.77	0.90	0.99	0.99	0.99	0.92
4	0.70	0.75	0.75	0.75	0.71	0.61	0.71	0.78	0.78	0.78	0.74
5	0.56	0.59	0.59	0.59	0.55	0.47	0.56	0.60	0.60	0.60	0.58

The computed demand-to-capacity ratio matrix in Exhibit 10-29 shows no segments with a v_d/c ratio greater than 1.0 in any time interval. Consequently, the facility is categorized as *globally undersaturated* and the analysis proceeds with computing the undersaturated service measures in Step 6a. Further, it is expected that no queuing will occur on the facility and that the volume served in each segment is identical to the input demand flows. Consequently, the matrix of volume-to-capacity ratios would be identical to the demand-to-capacity ratios in Exhibit 10-29. The resulting matrix of volumes served by segment and time interval is shown in Exhibit 10-30.

Exhibit 10-30
Volume-Served Matrix for Example Problem 1

Time Step	Volumes Served (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	4,505	4,955	4,955	4,955	4,685	5,225	4,865	5,315	5,315	5,315	5,045
2	4,955	5,495	5,495	5,495	5,135	5,855	5,495	6,035	6,035	6,035	5,765
3	5,225	5,855	5,855	5,855	5,585	6,395	6,035	6,665	6,665	6,665	6,215
4	4,685	5,045	5,045	5,045	4,775	5,135	4,775	5,225	5,225	5,225	4,955
5	3,785	3,965	3,965	3,965	3,695	3,965	3,785	4,055	4,055	4,055	3,875

Step 6a: Compute Undersaturated Segment Service Measures

Since the facility is globally undersaturated, the methodology proceeds to calculate service measures for each segment and each time period, starting with the first segment in Time Step 1. The computational details for each segment type are exactly as described in Chapters 11, 12, and 13. The weaving methodology in Chapter 13 checks whether the weaving short length L_S is less than or equal to the maximum weaving length L_{max}. It is assumed that, for any time interval where L_S is longer than L_{max}, the weaving segment will operate as a basic freeway segment.

The basic performance measures computed for each segment and each time step are the segment speed (Exhibit 10-31), density (Exhibit 10-32), and LOS (Exhibit 10-33).

Exhibit 10-31
Speed Matrix for Example Problem 1

Time Step	Speed (mi/h) by Segment 1	2	3	4	5	6	7	8	9	10	11
1	60.0	53.9	59.7	56.1	60.0	48.0	59.9	53.4	53.4	56.0	59.7
2	59.8	53.2	58.6	55.8	59.6	46.7	58.6	52.2	52.2	55.6	57.5
3	59.4	52.5	57.1	55.7	58.3	46.1	56.1	50.6	50.6	55.2	55.0
4	60.0	53.8	59.7	56.1	60.0	49.7	60.0	53.5	53.5	56.0	59.8
5	60.0	54.9	59.8	56.3	60.0	52.5	60.0	54.8	54.8	56.5	60.0

Exhibit 10-32
Density Matrix for Example Problem 1

Time Step	Density (veh/mi/ln) by Segment 1	2	3	4	5	6	7	8	9	10	11
1	25.0	30.7	27.7	29.4	26.0	27.2	27.1	33.2	33.2	31.7	28.2
2	27.6	34.5	31.3	32.8	28.7	31.3	31.3	38.5	38.5	36.2	33.4
3	29.3	37.2	34.2	35.0	31.9	34.6	35.8	43.9	43.9	40.3	37.7
4	26.0	31.3	28.2	30.0	26.5	25.8	26.5	32.5	32.5	31.1	27.6
5	21.0	24.1	22.1	23.5	20.5	18.9	21.0	24.7	24.7	23.9	21.5

Exhibit 10-33
LOS Matrix for Example Problem 1

Time Step	LOS by Segment 1	2	3	4	5	6	7	8	9	10	11
1	C	C	D	C	D	C	D	D	D	D	D
2	D	D	D	D	D	D	D	D	D	D	D
3	D	D	D	D	D	E	E	E	E	E	E
4	D	C	D	D	D	C	D	D	D	D	D
5	C	C	C	C	C	B	C	C	C	C	C

Step 7: Compute Facility Service Measures and Determine LOS

In the final analysis step, facilitywide performance and service measures are calculated for each time step. Example calculations are provided for the first time step only; summary results are shown for all five time steps.

First, the facility space mean speed S is calculated for time interval $t = 1$ from the 11 individual segment flows $SF(i, t)$, segment lengths $L(i)$, and space mean speeds in each segment and time period $U(i, t)$.

$$S(t=1) = \frac{\sum_{i=1}^{11} SF(i,1) \times L(i)}{\sum_{i=1}^{11} SF(i,1) \times \frac{L(i)}{U(i,1)}}$$

$$\sum_{i=1}^{11} SF(i,1) \times L(i) \quad \begin{aligned} &= 4{,}505 \times 5{,}280 + 4{,}955 \times 1{,}500 + 4{,}955 \times 2{,}280 + 4{,}955 \times 1{,}500 \\ &+ 4{,}685 \times 5{,}280 + 5{,}225 \times 2{,}640 + 4{,}865 \times 5{,}280 + 5{,}315 \times 1{,}140 \\ &+ 5{,}315 \times 360 + 5{,}315 \times 1{,}140 + 5{,}045 \times 5{,}280 \\ &= 154{,}836{,}000 \text{ veh-ft} \end{aligned}$$

$$\sum_{i=1}^{11} SF(i,1) \times \frac{L(i)}{U(i,1)} \quad \begin{aligned} &= (4{,}505 \times 5{,}280/60.00) + (4{,}955 \times 1{,}500/53.9) \\ &+ (4{,}955 \times 2{,}280/59.70) + (4{,}955 \times 1{,}500/56.10) \\ &+ (4{,}685 \times 5{,}280/60.00) + (5{,}225 \times 2{,}640/48.00) \\ &+ (4{,}865 \times 5{,}280/59.90) + (5{,}315 \times 1{,}140/53.40) \\ &+ (5{,}315 \times 360/53.40) + (5{,}315 \times 1{,}140/56.00) \\ &+ (5{,}045 \times 5{,}280/59.70) \\ &= 2{,}688{,}024 \text{ veh-ft/mi/h} \end{aligned}$$

$$S(t=1) = \frac{154{,}836{,}000}{2{,}688{,}024} = 57.6 \text{ mi/h}$$

Second, the average facility density is calculated for Time Step 1 from the individual segment densities D, segment lengths L, and number of vehicles in each segment N:

$$D(t=1)=\frac{\sum_{i=1}^{11} D(i,1)\times L(i)\times N(i,1)}{\sum_{i=1}^{11} L(i)N(i,1)}$$

$$\begin{aligned}\sum_{i=1}^{11} D(i,1)\times L(i)\times N(i,1) &= (25.0\times 5{,}280\times 3)+(30.7\times 1{,}500\times 3)+(27.7\times 2{,}280\times 3)\\ &+(29.4\times 1{,}500\times 3)+(26.0\times 5{,}280\times 3)+(27.2\times 2{,}640\times 4)\\ &+(27.1\times 5{,}280\times 3)+(33.2\times 1{,}140\times 3)+(33.2\times 360\times 3)\\ &+(31.7\times 1{,}140\times 3)+(28.2\times 5{,}280\times 3)\\ &=2{,}687{,}957\ \text{(veh/mi/ln)(ln-ft)}\end{aligned}$$

$$\begin{aligned}\sum_{i=1}^{11} L(i)N(i,1) &= (5{,}280\times 3)+(1{,}500\times 3)+(2{,}280\times 3)+(1{,}500\times 3)\\ &+(5{,}280\times 3)+(2{,}640\times 4)+(5{,}280\times 3)+(1{,}140\times 3)\\ &+(360\times 3)+(1{,}140\times 3)+(5{,}280\times 3)\\ &=97{,}680\ \text{ln-ft}\end{aligned}$$

$$D(t=1)=\frac{2{,}687{,}957}{97{,}680}=27.5\ \text{veh/mi/ln}$$

These calculations are repeated for all five time steps. The overall space mean speed across all time intervals is calculated as follows:

$$S(p=5)=\frac{\sum_{p=1}^{5}\sum_{i=1}^{11} SF(i,p)L(i)}{\sum_{p=1}^{5}\sum_{i=1}^{11} SF(i,p)\dfrac{L(i)}{U(i,p)}}$$

The overall average density across all time intervals is calculated as follows:

$$D(p=5)=\frac{\sum_{p=1}^{5}\sum_{i=1}^{11} D(i,p)\times L(i)\times N(i,p)}{\sum_{p=1}^{5}\sum_{i=1}^{11} L(i)N(i,p)}$$

The resulting performance and service measures for Time Steps 1–5 and the facility totals are shown in Exhibit 10-34. The LOS for each time interval is determined directly from the average density for each time interval by using Exhibit 10-7. No LOS is defined for the average across all time intervals.

Exhibit 10-34
Facility Performance Measure Summary for Example Problem 1

	Performance Measures		
Time Step	**Space Mean Speed (mi/h)**	**Average Density (veh/mi/ln)**	**LOS**
1	57.6	27.5	D
2	56.6	31.3	D
3	55.1	34.8	E
4	57.9	27.5	D
5	58.4	21.4	C
Total	**56.9**	**28.5**	—

Discussion

This facility turned out to be globally undersaturated. Consequently, the facility-aggregated performance measures could be calculated directly from the individual segment performance measures. An assessment of the segment service measures across the time–space domain can begin to highlight areas of potential congestion. Visually, this process can be facilitated by plotting the v_d/c, v_a/c, speed, or density matrices in contour plots.

EXAMPLE PROBLEM 2: EVALUATION OF AN OVERSATURATED FACILITY

The Facility

The facility used in Example Problem 2 is identical to the one in Example Problem 1, which is shown in Exhibit 10-25 and Exhibit 10-26.

The Facts

In addition to the information in Exhibit 10-25 and Exhibit 10-26, the following characteristics of the freeway facility are known:

Heavy vehicles = 5% trucks, 0% RVs (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp FFS = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for FFS = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level;

Analysis duration = 75 min (divided into five 15-min time steps); and

Demand adjustment = +11% increase in demand volumes across all segments and time steps compared with Example Problem 1.

Comments

The facility and all geometric inputs are identical to Example Problem 1. The same general comments apply. The results of Example Problem 1 suggested a globally undersaturated facility, but some segments were close to their capacity (v_d/c ratios approaching 1.0). In the second example, a facilitywide demand increase of 11% is applied to all segments and all time periods. Consequently, it is expected that parts of the facility may become oversaturated and that queues may form on the facility.

Step 1: Input Data

The revised traffic demand inputs for all 11 segments and five analysis intervals are shown in Exhibit 10-35.

Exhibit 10-35
Demand Inputs for Example Problem 2

Time Step (15 min)	Entering Flow Rate (veh/h)	Ramp Flow Rates by Time Period (veh/h)						Exiting Flow Rate (veh/h)
		ONR1	ONR2*	ONR3	OFR1	OFR2	OFR3	
1	5,001	500	599 (56)	500	600	400	300	5,600
2	5,500	599	799 (111)	599	400	400	300	6,399
3	5,800	699	899 (167)	699	300	400	500	6,899
4	5,200	400	400 (89)	500	300	400	300	5,500
5	4,201	200	300 (56)	300	300	200	200	4,301

* Numbers in parentheses indicate ONR-2 to OFR-2 demand flow rates in Weaving Segment 6.

The values in Exhibit 10-35 represent the adjusted demand flows on the facility as determined from field observations or demand projections. The actual volume served in each segment will be determined during application of the methodology and is expected to be less downstream of a congested segment. The demand flows are given for the extended time–space domain, consistent with this chapter's methodology. Peaking occurs in the third 15-min period. Since inputs are in the form of 15-min observations, no peak hour factor adjustment is necessary. Additional geometric and traffic-related inputs are as specified in Exhibit 10-25 and the facts section of the problem statement.

Step 2: Demand Adjustments

The traffic flows in Exhibit 10-35 have already been given in the form of actual demands and no further demand adjustments are necessary.

Step 3: Compute Segment Capacities

Since no changes to segment geometry were made, the segment capacities for basic and ramp segments are consistent with Example Problem 1 and Exhibit 10-28. Capacities for weaving segments are a function of weaving flow patterns, and the increased demand flows resulted in slight changes as shown in Exhibit 10-36.

Exhibit 10-36
Segment Capacities for Example Problem 2

Time Step	Capacities (veh/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1						8,253					
2						8,260					
3	6,732	6,732	6,732	6,732	6,732	8,303	6,732	6,732	6,732	6,732	6,732
4						8,382					
5						8,443					

Step 4: Adjust Segment Capacities

No capacity adjustments are made in this example.

Step 5: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios in Exhibit 10-37 are calculated from the demand flows in Exhibit 10-35 and from the segment capacities in Exhibit 10-36.

Exhibit 10-37
Segment Demand-to-Capacity Ratios for Example Problem 2

Time Step	Demand-to-Capacity Ratios by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	0.74	0.82	0.82	0.82	0.77	0.70	0.80	0.88	0.88	0.88	0.83
2	0.82	0.91	0.91	0.91	0.85	0.79	0.91	1.00	1.00	1.00	0.95
3	0.86	0.97	0.97	0.97	0.92	0.85	1.00	**1.10**	**1.10**	**1.10**	**1.02**
4	0.77	0.83	0.83	0.83	0.79	0.68	0.79	0.86	0.86	0.86	0.82
5	0.62	0.65	0.65	0.65	0.61	0.52	0.62	0.67	0.67	0.67	0.64

The computed v_d/c matrix in Exhibit 10-37 shows that Segments 8–11 now have v_d/c ratios greater than 1.0 (bold values). Consequently, the facility is categorized as *oversaturated* and the analysis proceeds with computing the oversaturated service measures in Step 6b. Further, it is expected that queuing will occur on the facility upstream of the congested segments and that the volume served in each segment downstream of the congested segments will be less than the demand. This residual demand will be served in later time intervals, provided that upstream demand drops and queues are allowed to clear.

Step 6b: Compute Oversaturated Segment Service Measures

The oversaturated computations apply to any segment with a v_d/c ratio greater than 1.0 as well as any segments upstream of those segments that experience queuing as a result of the bottleneck. All remaining segments are analyzed by using the individual segment methodologies of Chapters 11, 12, and 13, as applicable, with the caveat that volumes served may differ from demand flows.

Similar to Example Problem 1, the methodology calculates performance measures for each segment and each time period, starting with the first segment in Time Step 1. The computations are repeated for all segments for Time Steps 1 and 2 without encountering a segment with $v_d/c > 1.0$. Once the methodology enters Time Period 3 and Segment 8, the oversaturated computational module is invoked.

As the first active bottleneck, the v_d/c ratio for Segment 8 will be exactly 1.0 and will process traffic at its capacity. Consequently, demand for all downstream segments will be metered by that bottleneck. The unsatisfied demand is stored in upstream segments, which causes queuing in Segment 7 and perhaps further upstream segments depending on the level of excess demand. The rate of growth of the vehicle queue (wave speed) is estimated from shock wave theory, as discussed in detail in Chapter 25, Freeway Facilities: Supplemental. The performance measures (speed and density) of any segment with queuing are recomputed as discussed in Chapter 25, and the newly calculated values override the results from the segment-specific procedures.

Any unsatisfied demand is serviced in later time periods. As a result, volumes served in later time periods may be higher than the period demand flows. The resulting matrix of volumes served for Example Problem 2 is shown in Exhibit 10-38. The table emphasizes cells where volumes served are less than demand flows (in **bold**) and where volumes served are greater than demand flows (*italicized).*

Exhibit 10-38
Volume-Served Matrix for Example Problem 2

Time Step	Volumes Served (veh/h) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	5,001	5,500	5,500	5,500	5,200	5,800	5,400	5,900	5,900	5,900	5,600
2	5,500	6,099	6,099	6,099	5,700	6,400	6,099	6,699	6,699	6,699	6,399
3	5,800	6,499	6,499	6,499	**6,111**	**6,625**	**6,032**	**6,732**	**6,732**	**6,732**	**6,277**
4	5,200	5,600	5,600	5,600	*5,389*	*6,173*	*5,967*	*6,466*	*6,466*	*6,466*	*6,121*
5	4,201	4,401	4,401	4,401	4,101	4,401	4,201	4,501	4,501	4,501	4,301

As a result of the bottleneck activation in Segment 8 in Time Period 3, queues form in upstream Segments 7, 6, and 5. The queuing is associated with reduced speeds and increased densities in those segments. Details on how these measures are calculated for oversaturated segments are given in Chapter 25. The results in this chapter were obtained from the FREEVAL-2010 engine. The resulting performance measures computed for each segment and time interval are the speed (Exhibit 10-39), density (Exhibit 10-40), and LOS (Exhibit 10-41).

Exhibit 10-39
Speed Matrix for Example Problem 2

Time Step	Speed (mi/h) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	59.8	53.1	58.6	55.9	59.4	46.8	58.9	52.5	52.5	55.7	58.2
2	58.6	52.1	55.7	55.5	57.8	45.4	55.7	50.5	50.5	55.3	53.8
3	57.4	51.0	53.0	55.4	53.6	28.2	34.8	50.2	50.2	55.1	54.6
4	59.4	53.0	58.2	55.8	49.9	39.2	53.9	51.2	51.2	55.3	55.6
5	60.0	54.5	59.7	56.2	60.0	51.7	60.0	54.4	54.4	56.3	60.0

Exhibit 10-40
Density Matrix for Example Problem 2

Time Step	Density (veh/mi/ln) by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	27.9	34.5	31.3	32.8	29.2	31.0	30.6	37.5	37.5	35.3	32.1
2	31.3	39.0	36.5	36.7	32.9	35.8	36.5	44.2	44.2	40.4	39.7
3	33.7	42.5	40.9	39.1	38.0	**58.8**	**57.7**	44.7	44.7	40.7	38.3
4	29.2	35.2	32.1	33.4	36.0	39.4	36.9	42.1	42.1	38.9	36.7
5	23.3	26.9	24.6	26.1	22.8	21.3	23.3	27.6	27.6	26.6	23.9

Exhibit 10-41
Expanded LOS Matrix for Example Problem 2

Time Step	Density-Based LOS by Segment										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1	D	D	D	D	D	D	D	D	D	D	D
2	D	D	E	D	D	E	E	E	E	E	E
3	D	D	E	D	E	**F**	**F**	E	E	E	E
4	D	D	D	D	E	E	E	D	D	D	E
5	C	C	C	C	C	C	C	C	C	C	C
Time Step	**Demand-Based LOS by Segment**										
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**
1											
2											
3								**F**	**F**	**F**	**F**
4											
5											

The LOS table for oversaturated facilities (Exhibit 10-41) distinguishes between the conventional density-based LOS and a segment demand-based LOS. The density-based stratification strictly depends on the prevailing average density on each segment. Segments downstream of the bottleneck, whose capacities are greater than or equal to the bottleneck capacity, operate at LOS E (or better), even though their v_d/c ratios were greater than 1.0. The demand-based LOS identifies those segments with demand-to-capacity ratios exceeding 1.0 as if they had been evaluated in isolation (i.e., using methodologies of Chapters 11, 12, and 13). By contrasting the two parts of the LOS table, the analyst can develop an understanding of the metering effect of the bottleneck.

Step 7: Compute Facility Service Measures and Determine LOS

In the final analysis step, facilitywide performance and service measures are calculated for each time interval (Exhibit 10-42), consistent with Example Problem 1. Only summary results are shown in this case, since the computations have already been shown. The facility operates at LOS F in Time Period 3, since one or more individual segments have *d/c* ratios ≥ 1.0, even though the average facility density is below the LOS F threshold.

Exhibit 10-42
Facility Performance Measure Summary for Example Problem 2

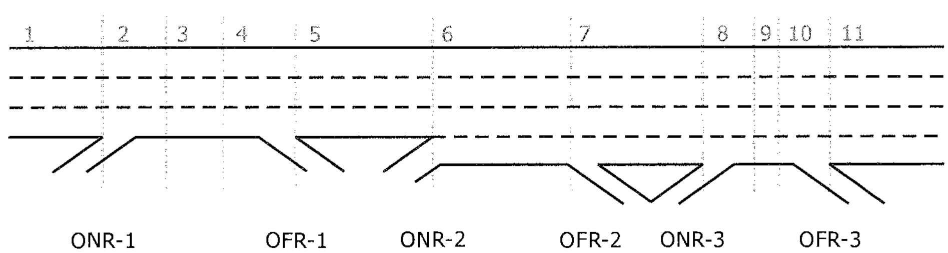
Time Interval	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	56.7	31.0	D
2	54.5	36.1	E
3	46.3	43.7	F
4	52.8	35.4	E
5	58.2	23.8	C
Total	**52.9**	**34.0**	—

EXAMPLE PROBLEM 3: CAPACITY IMPROVEMENTS TO AN OVERSATURATED FACILITY

The Facility

In this example, portions of the congested facility in Example Problem 2 are being improved in an attempt to alleviate the congestion resulting from the Segment 8 bottleneck. Exhibit 10-43 shows the upgraded facility geometry.

Exhibit 10-43
Freeway Facility in Example Problem 3



The modified geometry of the 6-mi directional freeway facility is reflected in Exhibit 10-44.

Exhibit 10-44
Geometry of Directional Freeway Facility in Example Problem 3

Segment No.	1	2	3	4	5	6	7	8	9	10	11
Segment type	B	ONR	B	OFR	B	B or W	B	ONR	R	OFR	B
Segment length (ft)	5,280	1,500	2,280	1,500	5,280	2,640	5,280	1,140	360	1,140	5,280
No. of lanes	3	3	3	3	3	4	**4**	**4**	**4**	**4**	**4**

Note: B = basic freeway segment, W = weaving segment, ONR = on-ramp (merge) segment, OFR = off-ramp (diverge) segment, R = overlapping ramp segment.
Bold type indicates geometry changes from Example Problems 1 and 2.

The facility improvements consisted of adding a lane to Segments 7–11 to give the facility a continuous four-lane cross section starting in Segment 6. While the active bottleneck in Example Problem 2 was in Segment 8, the prior analysis showed that other segments (Segments 9–11) showed similar demand-to-capacity ratios greater than 1.0. Consequently, any capacity improvements that are limited to Segment 8 would have merely moved the spatial location of the bottleneck further downstream rather than improving the overall facility. Segments 9–11 may also be referred to as "hidden" or "inactive" bottlenecks, because their predicted congestion is mitigated by the upstream metering of traffic.

The Facts

In addition to the information contained in Exhibit 10-43 and Exhibit 10-44, the following characteristics of the freeway facility are known:

Heavy vehicles = 5% trucks, 0% RVs (all movements);

Driver population = regular commuters;

FFS = 60 mi/h (all mainline segments);

Ramp FFS = 40 mi/h (all ramps);

Acceleration lane length = 500 ft (all ramps);

Deceleration lane length = 500 ft (all ramps);

D_{jam} = 190 pc/mi/ln;

c_{IFL} = 2,300 pc/h/ln (for FFS = 60 mi/h);

L_s = 1,640 ft (for Weaving Segment 6);

TRD = 1.0 ramp/mi;

Terrain = level;

Analysis duration = 75 min (divided into five 15-min intervals); and

Demand adjustment = +11% (all segments and all time intervals).

Comments

The traffic demand flow inputs are identical to those in Example Problem 2, which reflected an 11% increase in traffic applied to all segments and all time periods. In an attempt to solve the congestion effect found in the earlier example, the facility was widened in Segments 7 and 11. This change directly affects the capacities of those segments.

In a more subtle way, the proposed modifications also change some of the defining parameters of Weaving Segment 6 as well. With the added continuous lane downstream of the segment, the required number of lane changes from the ramp to the freeway is reduced from one to zero, following the guidelines in Chapter 12. These changes need to be considered when the undersaturated performance of that segment is evaluated. The weaving segment's capacity is unchanged relative to Example Problem 2, since, even with the proposed improvements, the number of weaving lanes remains two.

Step 1: Input Data

Traffic demand inputs for all 11 segments and five analysis intervals are identical to those in Example Problem 2 as shown in Exhibit 10-35. The values in Exhibit 10-35 represent the adjusted demand flows on the facility as determined from field observations or other sources. The actual volume served in each segment will be determined during the methodologies and is expected to be less downstream of a congested segment. Additional geometric and traffic-related inputs are as specified in Exhibit 10-44 and the facts section of the problem statement.

Step 2: Demand Adjustments

The traffic flows in Exhibit 10-35 have already been given in the form of actual demands and no further demand adjustments are necessary.

Step 3: Compute Segment Capacities

Segment capacities are determined by using the methodologies of Chapter 11 for basic freeway segments, Chapter 12 for weaving segments, and Chapter 13 for merge and diverge segments. The resulting capacities are shown in Exhibit 10-45. Since the capacity of a weaving segment depends on traffic patterns, it varies by time period. The remaining capacities are constant for all five time steps. The capacities for Segments 1–5 and for Segments 7–11 are the same, since the segments have the same basic cross section.

Exhibit 10-45 Segment Capacities for Example Problem 3

Time	Capacities (veh/h) by Segment										
Step	1	2	3	4	5	6	7	8	9	10	11
1						8,253					
2						8,260					
3	6,732	6,732	6,732	6,732	6,732	8,303	8,976	8,976	8,976	8,976	8,976
4						8,382					
5						8,443					

Step 4: Adjust Segment Capacities

No additional capacity adjustments are made in this example.

Step 5: Compute Demand-to-Capacity Ratios

The demand-to-capacity ratios are calculated from the demand flows in Exhibit 10-35 and segment capacities in Exhibit 10-45.

Exhibit 10-46 Segment Demand-to-Capacity Ratios for Example Problem 3

Time	Demand-to-Capacity Ratio by Segment										
Step	1	2	3	4	5	6	7	8	9	10	11
1	0.74	0.82	0.82	0.82	0.77	0.70	0.60	0.66	0.66	0.66	0.62
2	0.82	0.91	0.91	0.91	0.85	0.79	0.68	0.75	0.75	0.75	0.71
3	0.86	0.97	0.97	0.97	0.92	0.85	0.75	0.82	0.82	0.82	0.77
4	0.77	0.83	0.83	0.83	0.79	0.68	0.59	0.65	0.65	0.65	0.61
5	0.62	0.65	0.65	0.65	0.61	0.52	0.47	0.50	0.50	0.50	0.48

The demand-to-capacity ratio matrix for Example Problem 3 (Exhibit 10-46) shows that the capacity improvements successfully reduced all the previously congested segments to $v_d/c < 1.0$. Therefore, it is expected that the facility will operate as *globally undersaturated* and that all segment performance measures can be directly computed by using the methodologies in Chapters 11, 12, and 13 in Step 6a.

Step 6a: Compute Undersaturated Segment Service Measures

Since the facility is globally undersaturated, the methodology proceeds to calculate performance and service measures for each segment and each time step, starting with the first segment in Time Interval 1. The computational details for each segment type are exactly as described in Chapters 11, 12, and 13. The weaving methodology in Chapter 13 checks whether the weaving short length L_S is less than or equal to the maximum weaving length L_{max}. It is assumed that, for any time interval where L_S is longer than L_{max}, the weaving segment will operate as a basic freeway segment.

The basic performance service measures computed for each segment and each time interval include the segment speed (Exhibit 10-47), density (Exhibit 10-48), and LOS (Exhibit 10-49).

Exhibit 10-47
Speed Matrix for Example Problem 3

Time Step	Speed (mi/h) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	59.8	53.1	58.6	55.9	59.4	50.4	60.0	54.9	54.9	58.1	60.0
2	58.6	52.1	55.7	55.5	57.8	50.0	60.0	54.3	54.3	57.7	60.0
3	57.4	51.0	53.0	55.4	55.1	49.7	59.8	53.6	53.6	57.2	59.5
4	59.4	53.0	58.2	55.8	59.2	50.7	60.0	55.0	55.0	58.1	60.0
5	60.0	54.5	59.7	56.2	60.0	53.4	60.0	55.9	55.9	58.8	60.0

Exhibit 10-48
Density Matrix for Example Problem 3

Time Step	Density (veh/mi/ln) by Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	27.9	34.5	31.3	32.8	29.2	28.8	22.5	26.9	26.9	25.4	23.3
2	31.3	39.0	36.5	36.7	32.9	32.5	25.4	30.9	30.9	29.0	26.7
3	33.7	42.5	40.9	39.1	37.5	35.7	28.0	34.5	34.5	32.4	29.0
4	29.2	35.2	32.1	33.4	29.8	28.1	22.1	26.4	26.4	24.9	22.9
5	23.3	26.9	24.6	26.1	22.8	20.6	17.5	20.1	20.1	19.1	17.9

Exhibit 10-49
LOS Matrix for Example Problem 3

Time Step	LOS for Segment										
	1	2	3	4	5	6	7	8	9	10	11
1	D	D	D	D	D	D	C	C	C	C	C
2	D	D	E	D	D	D	D	C	C	C	D
3	D	D	E	D	E	E	D	D	D	D	D
4	D	D	D	D	D	D	C	C	C	C	C
5	C	C	C	C	C	C	B	B	B	B	C

Step 7: Compute Facility Service Measures and Determine LOS

In the final analysis step, facilitywide performance and service measures are calculated for each time step (Exhibit 10-50), consistent with Example Problem 2. Only summary results are shown in this case, since the computations have already been shown. The improvement has been able to restore the facility LOS to the values experienced in the original pregrowth scenario shown in Exhibit 10-34.

Exhibit 10-50
Facility Performance Measure Summary for Example Problem 3

Time Step	Performance Measure: Space Mean Speed (mi/h)	Performance Measure: Average Density (veh/mi/ln)	LOS
1	57.9	26.8	D
2	57.1	30.4	D
3	56.0	33.5	D
4	57.8	26.9	D
5	58.6	20.8	C
Total	**57.3**	**27.7**	—

5. REFERENCES

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

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CHAPTER 11
BASIC FREEWAY SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Basic freeway segments are defined as those freeway segments that are outside the influence of merging, diverging, or weaving maneuvers. In general, this means that lane-changing activity is not significantly influenced by the presence of ramps and weaving segments. Lane-changing activity primarily reflects the normal desire of drivers to optimize their efficiency through lane-changing and passing maneuvers.

A complete discussion of influence areas is included in Chapter 10, Freeway Facilities, with additional discussion in Chapters 12, Freeway Weaving Segments, and 13, Freeway Merge and Diverge Segments. In general terms, the influence area of merge (on-ramp) segments extends for 1,500 ft downstream of the merge point; the influence area of diverge (off-ramp) segments extends for 1,500 ft upstream of the diverge point; and the influence area of weaving segments extends 500 ft upstream and downstream of the segment itself. This description is not to suggest that the influence of these segments cannot extend over a broader range, particularly under breakdown conditions. Under stable operations, however, these distances define the areas most affected by merge, diverge, and weaving movements. The impact of breakdowns in any type of freeway segment on adjacent segments can be addressed by using the methodology of Chapter 10, Freeway Facilities.

Chapter 11, Basic Freeway Segments, provides a methodology for analyzing the capacity and level of service (LOS) of existing or planned basic freeway segments. The methodology can also be used for design applications, where the number of lanes needed to provide a target LOS for an existing or projected demand flow rate can be found.

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

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

BASE CONDITIONS

Base conditions include good weather and visibility and no incidents or accidents. These conditions are always assumed to exist.

The base conditions under which the full capacity of a basic freeway 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. This chapter's methodology assumes that these conditions exist. If any of these conditions do not exist, the speed, LOS, and capacity of the freeway segment can be expected to be worse than those predicted by this methodology.

Base conditions also include the following conditions, which can be adjusted as the methodology is applied to address situations in which these conditions do not exist:

Base conditions also include 0% heavy vehicles, a driver population composed of regular users of the freeway, and 12-ft lane widths and minimum 6-ft right-side clearances.

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

- No heavy vehicles [trucks, buses, recreational vehicles (RVs)] in the traffic stream;
- A driver population composed primarily of regular users who are familiar with the facility; and
- Minimum 12-ft lane widths and 6-ft right-side clearances.

FLOW CHARACTERISTICS UNDER BASE CONDITIONS

Traffic flow within basic freeway segments can be highly varied depending on the conditions constricting flow at upstream and downstream bottleneck locations. Such bottlenecks can be created by merging, diverging, or weaving traffic; lane drops; maintenance and construction activities; traffic accidents or incidents; objects in the roadway; or all of the foregoing. 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.

Types of Flow

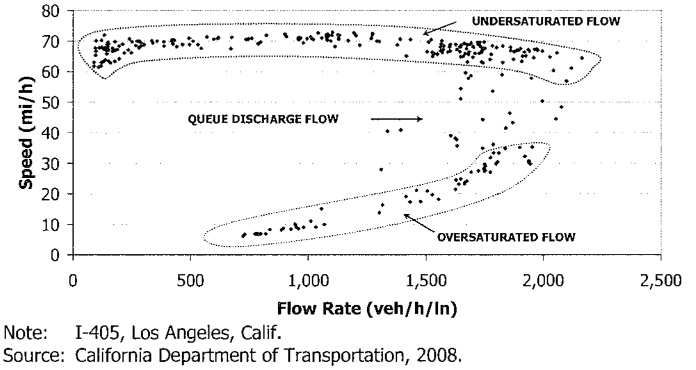
Chapter 2 describes in more detail the types of traffic flow on basic freeway segments.

As was discussed in more detail in Chapter 2, Applications, traffic flow within a basic freeway 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 traffic flow that has just passed through a bottleneck and is accelerating back to drivers' desired speeds for the prevailing conditions. As long as another downstream bottleneck does not exist, queue discharge flow is 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 site 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 11-1, using data from a freeway in California.

Exhibit 11-1
Three Types of Freeway Flow



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

The basic freeway segment methodology is based on undersaturated flow conditions.

The analysis methodology for basic freeway segments is based entirely on calibrations of the speed–flow relationships under base conditions with undersaturated flow. The methodology identifies cases in which failure has occurred but does not attempt to describe operating conditions when a segment has failed. The methodology of Chapter 10, Freeway Facilities, should be used for oversaturated conditions.

Speed–Flow Curves for Base Conditions

A set of speed–flow curves for basic freeway segments operating under base conditions is shown in Exhibit 11-2. There are five curves, one for each of five levels of free-flow speed (FFS): 75 mi/h, 70 mi/h, 65 mi/h, 60 mi/h, and 55 mi/h. Technically speaking, the FFS is the speed at the y-intercept of each curve. In practical terms, there are two ranges in the shape of the curves:

- For each curve, a range of flows exists from 0 pc/h/ln to a breakpoint in which speed remains constant at the FFS. The ranges vary for each of the curves as follows:

  FFS = 75 mi/h: 0–1,000 pc/h/ln;

  FFS = 70 mi/h: 0–1,200 pc/h/ln;

  FFS = 65 mi/h: 0–1,400 pc/h/ln;

  FFS = 60 mi/h: 0–1,600 pc/h/ln;

  FFS = 55 mi/h: 0–1,800 pc/h/ln.

- At flow rates above the breakpoint of each curve, speeds decline at an increasing rate until capacity is reached.

Exhibit 11-2
Speed–Flow Curves for Basic Freeway Segments Under Base Conditions

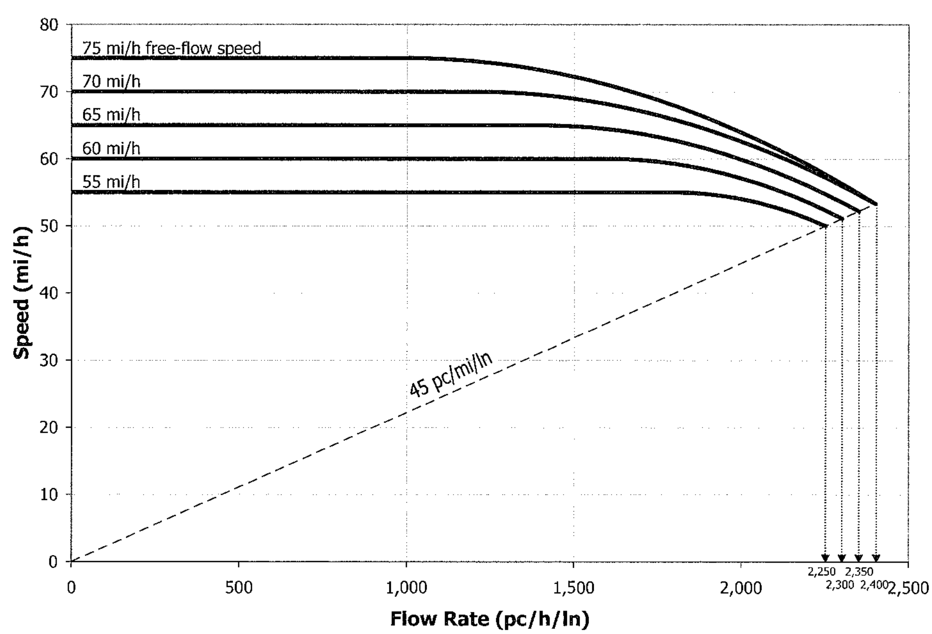


Exhibit 11-3 shows the equations that define each of the curves in Exhibit 11-2. Because estimating or measuring FFS is difficult, and there is considerable variation in observed and predicted values, no attempt should be made to

FFS should be rounded to the nearest 5 mi/h.

interpolate between the basic curves. FFS should be rounded to the nearest 5 mi/h as follows:

- ≥72.5 mi/h < 77.5 mi/h: use FFS = 75 mi/h,
- ≥67.5 mi/h < 72.5 mi/h: use FFS = 70 mi/h,
- ≥62.5 mi/h < 67.5 mi/h: use FFS = 65 mi/h,
- ≥57.5 mi/h < 62.5 mi/h: use FFS = 60 mi/h,
- ≥52.5 mi/h < 57.5 mi/h: use FFS = 55 mi/h.

Exhibit 11-3
Equations Describing Speed–Flow Curves in Exhibit 11-2 (Speeds in mi/h)

FFS (mi/h)	Breakpoint (pc/h/ln)	Flow Rate Range: ≥0 ≤ Breakpoint	Flow Rate Range: >Breakpoint ≤ Capacity
75	1,000	75	$75 - 0.00001107\,(v_p - 1{,}000)^2$
70	1,200	70	$70 - 0.00001160\,(v_p - 1{,}200)^2$
65	1,400	65	$65 - 0.00001418\,(v_p - 1{,}400)^2$
60	1,600	60	$60 - 0.00001816\,(v_p - 1{,}600)^2$
55	1,800	55	$55 - 0.00002469\,(v_p - 1{,}800)^2$

Notes: FFS = free-flow speed, v_p = demand flow rate (pc/h/ln) under equivalent base conditions.
Maximum flow rate for the equations is capacity: 2,400 pc/h/ln for 70- and 75-mph FFS; 2,350 pc/h/ln for 65-mph FFS; 2,300 pc/h/ln for 60-mph FFS; and 2,250 pc/h/ln for 55-mph FFS.

The research leading to these curves (*1*, *2*) found that several factors affect the FFS of a basic freeway segment, including the lane width, right-shoulder clearance, and ramp density. Ramp density is the average number of on-ramps plus off-ramps in a 6-mi range, 3 mi upstream and 3 mi downstream of the midpoint of the study segment. Many other factors are likely to influence FFS: horizontal and vertical alignment, posted speed limits, level of speed enforcement, lighting conditions, and weather. Although these factors may affect FFS, little information is available that would allow their quantification.

CAPACITY UNDER BASE CONDITIONS

The capacity of a basic freeway segment under base conditions varies with the FFS. For 70- and 75-mi/h FFS, the capacity is 2,400 pc/h/ln. For lesser levels of FFS, capacity diminishes slightly. For 65-mi/h FFS, the capacity is 2,350 pc/h/ln; for 60-mi/h FFS, 2,300 pc/h/ln; and for 55-mi/h FFS, 2,250 pc/h/ln.

Base capacity values refer to the average flow rate across all lanes. Individual lanes could have stable flows in excess of these values.

Since freeways usually do not operate under base conditions, observed capacity values will typically be lower than the base capacity values.

Chapter 10, Freeway Facilities, contains information that would allow these values to be reduced to reflect long- and short-term construction and maintenance activities, adverse weather conditions, and accidents or incidents.

These values represent national norms. It should be remembered that capacity varies stochastically and that any given location could have a larger or smaller value. It should also be remembered that 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.

As shown in Exhibit 11-2, it is believed that basic freeway segments reach capacity at a density of approximately 45 passenger cars per mile per lane (pc/mi/ln), which may vary slightly from location to location. At this density, vehicles are too closely spaced to dampen the impact of any perturbation in flow, such as a lane change or a vehicle entering the freeway, without causing a disruption that propagates upstream.

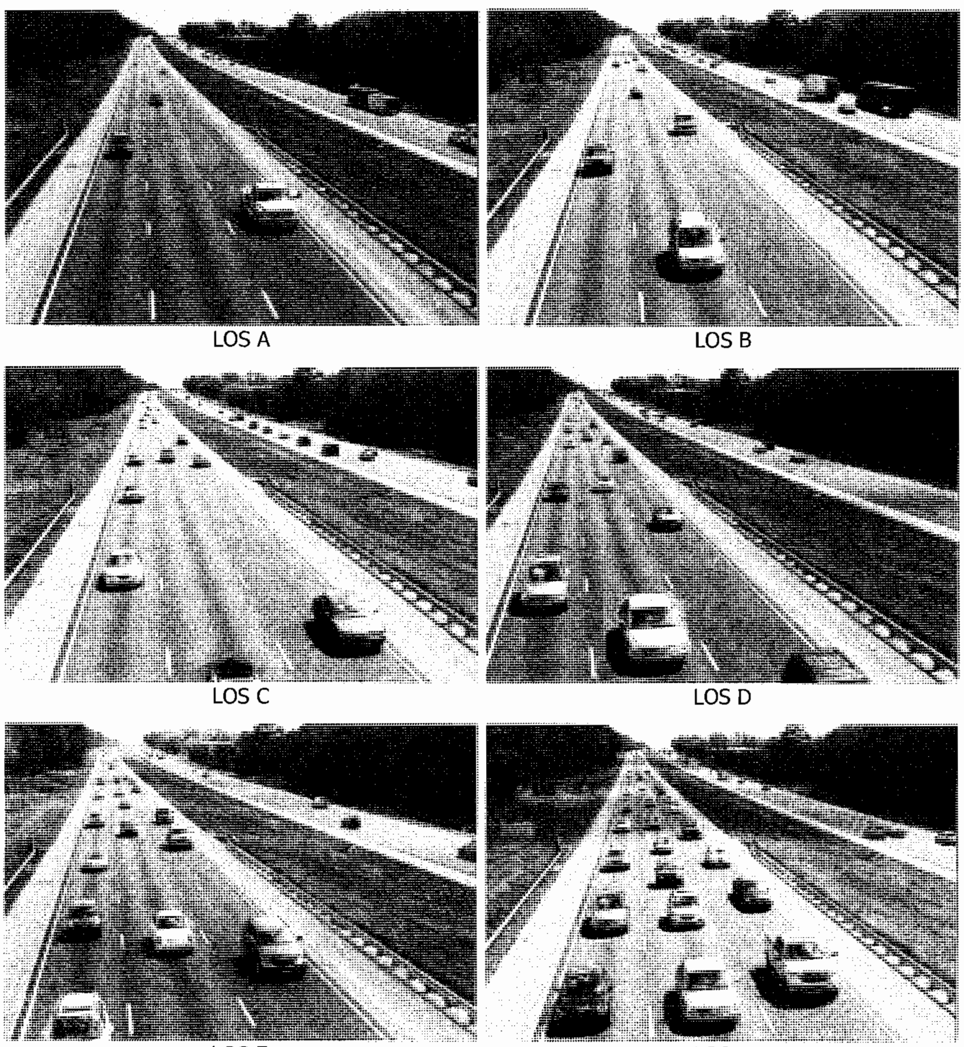
LOS FOR BASIC FREEWAY SEGMENTS

LOS for basic freeway segments is defined by density.

LOS on a basic freeway segment is defined by density. Although speed is a major concern of drivers as related to service quality, it would be difficult to describe LOS by using speed, since it remains constant up to flow rates of 1,000 to 1,800 pc/h/ln, depending on the FFS. Density describes the proximity to other vehicles and is related to the freedom to maneuver within the traffic stream. Unlike speed, however, density is sensitive to flow rates throughout the range of flows.

Exhibit 11-4 visually demonstrates the six LOS defined for basic freeway segments. LOS are defined to represent reasonable ranges in the three critical flow variables: speed, density, and flow rate.

Exhibit 11-4
LOS Examples



LOS A
LOS B
LOS C
LOS D
LOS E
LOS F

Freeway LOS Described

LOS A describes free-flow operations. FFS prevails on the freeway, 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 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 and point breakdowns are still easily absorbed.

LOS C provides for flow with speeds near the FFS of the freeway. 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 capacity. Operations on the freeway 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 a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic flow. At capacity, 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.

LOS F describes breakdown, or 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.

Breakdown (LOS F) occurs whenever the demand-to-capacity ratio exceeds 1.00.

In all cases, breakdown occurs when the ratio of existing demand to actual capacity, or of forecast demand to estimated capacity, exceeds 1.00. Operations immediately downstream of, or even at, such a point, however, are generally at or near LOS E, and downstream operations improve (assuming that there are no additional downstream bottlenecks) as discharging vehicles move away from the bottleneck.

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 v/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.

The effects of a breakdown may extend upstream for a considerable distance.

LOS Criteria

A basic freeway segment can be characterized by three performance measures: density in passenger cars per mile per lane (pc/mi/ln), space mean speed in miles per hour (mi/h), 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 segments is density. Exhibit 11-5 shows the criteria.

Exhibit 11-5
LOS Criteria for Basic Freeway Segments

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

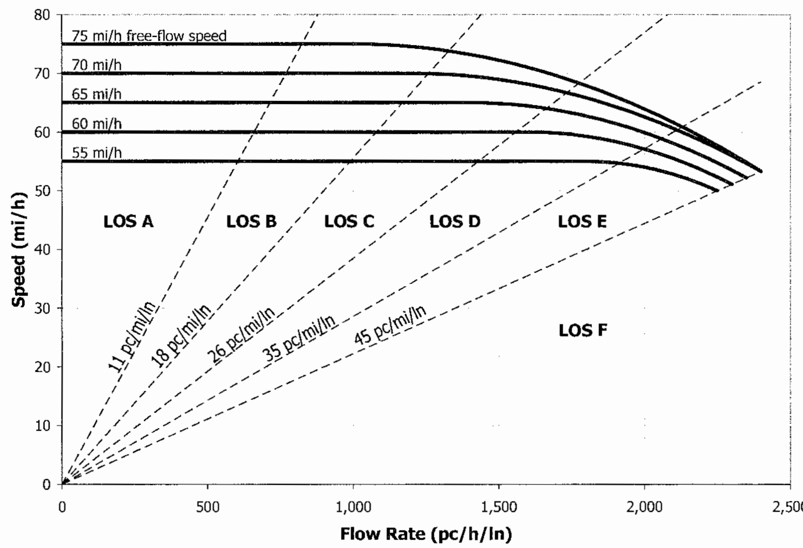
For all LOS, the density boundaries on basic freeway segments are the same as those for surface multilane highways, except at the LOS E–F boundary. Traffic characteristics are such that the maximum flow rates at any given LOS are lower on multilane highways than on similar basic freeway segments.

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 Highway Capacity and Quality of Service Committee. The upper value shown for LOS F (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 11-2, 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 analytic 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, must be applied to determine a more precise density for such cases.

Exhibit 11-6 illustrates the defined LOS on the base speed–flow curves. On a speed–flow plot, density is a line of constant slope beginning at the origin. The LOS boundaries were defined to produce reasonable ranges within each LOS on these speed–flow relationships.

Exhibit 11-6
LOS for Basic Freeway Segments



REQUIRED INPUT DATA

The analysis of a basic freeway segment requires details concerning the geometric characteristics of the segment and the demand characteristics of the users of the segment. This section presents the required input data for the basic freeway segment methodology; specifics about individual parameters are given in the Methodology section.

Freeway Data

The following information on the segment's geometric features is needed to conduct an analysis (typical ranges for these parameters are shown):

1. FFS: 55 to 75 mi/h;
2. Number of mainline freeway lanes (one direction): at least two;
3. Lane width: 10 ft to 12 ft or more;
4. Right-side lateral clearance: 0 ft to more than 6 ft;
5. Total ramp density: 0 to 6 ramps/mi; and
6. Terrain: level, rolling, or mountainous, or specific length and percent grade.

Demand Data

The following information on the segment's users is required:

1. Demand during the analysis hour or daily demand and *K*- and *D*-factors;
2. Heavy-vehicle presence (proportion of trucks, buses, and RVs): 0 to 100% in general terrain, or 0 to 25% or more for specific grades;
3. Peak-hour factor (PHF): up to 1.00; and
4. Driver population factor: 0.85 to 1.00.

Length of Analysis Period

The analysis period for any freeway analysis is generally the peak 15-min period within the peak hour. Any 15-min period can be analyzed, however.

2. METHODOLOGY

This chapter's methodology can be used to analyze the capacity, LOS, lane requirements, and effects of design features on the performance of basic freeway segments. The methodology is based on the results of an NCHRP study (*1*), which has been partially updated (*2*). A number of significant publications were also used in the development of the methodology (*3–12*).

LIMITATIONS OF METHODOLOGY

This chapter's methodology does not apply to or take into account (without modification by the analyst) the following:

- Special lanes reserved for a single vehicle type, such as high-occupancy-vehicle (HOV) lanes, truck lanes, and climbing lanes;
- Lane control (to restrict lane changing);
- Extended bridge and tunnel segments;
- Segments near a toll plaza;
- Facilities with FFS less than 55 mi/h or more than 75 mi/h;
- The influence of downstream queuing on a segment;
- Posted speed limit and enforcement practices;
- Presence of intelligent transportation systems (ITS) related to vehicle or driver guidance;
- Capacity-enhancing effects of ramp metering;
- Operational effects of oversaturated conditions; and
- Operational effects of construction operations.

In most of the cases just cited, the analyst would have to utilize alternative tools or draw on other research information and develop special-purpose modifications of this methodology to incorporate the effects of any of the cited conditions. Operational effects of oversaturated conditions, incidents, work zones, and weather and lighting conditions can be evaluated with the methodology of Chapter 10, Freeway Facilities. Operational effects of active traffic management measures are discussed in Chapter 35.

Active traffic management measures for freeways discussed in Chapter 35 consist of

- *Dynamic demand metering,*
- *Congestion pricing,*
- *Traveler information systems,*
- *Dynamic lane and shoulder management,*
- *Speed harmonization,*
- *Incident management, and*
- *Work zone traffic management.*

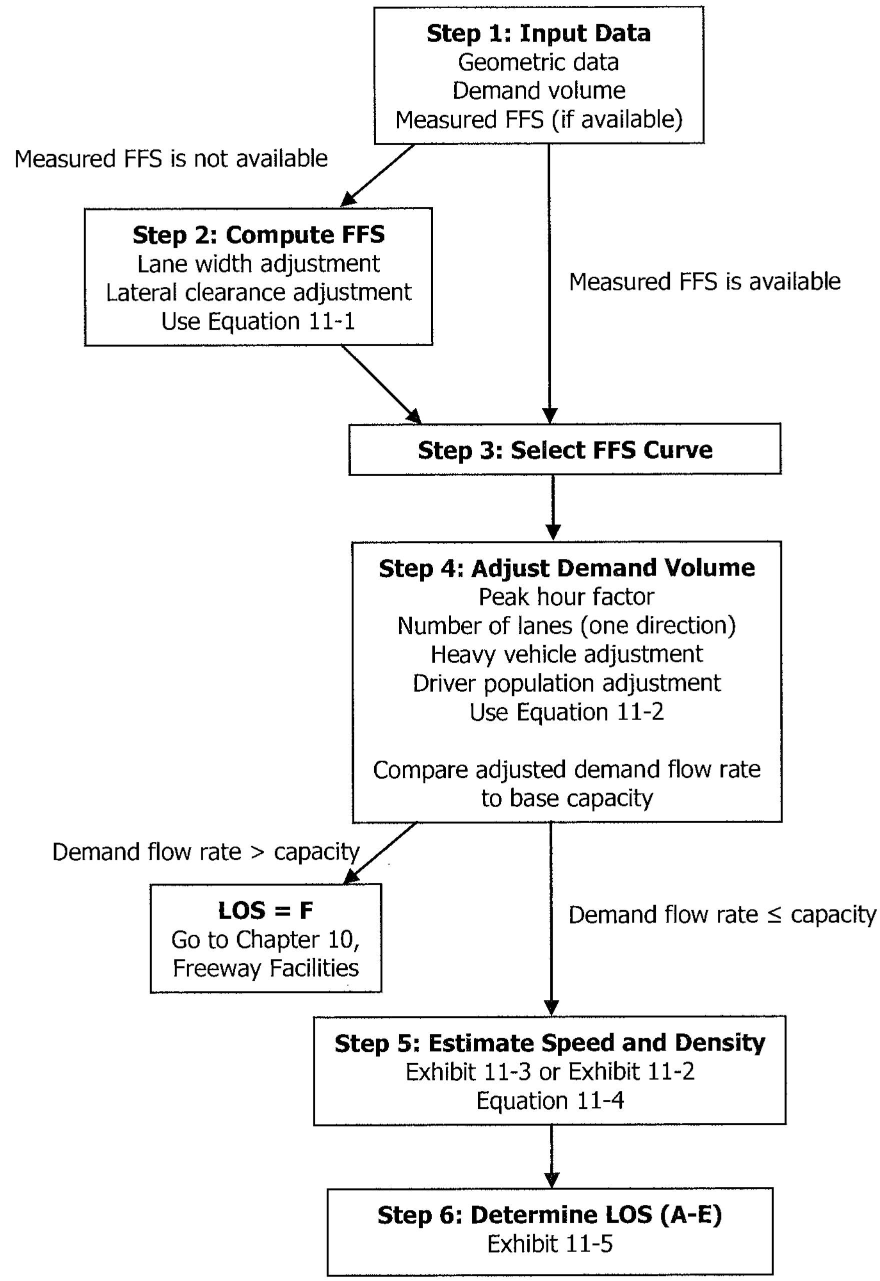
OVERVIEW OF METHODOLOGY

The methodology of this chapter is for the analysis of basic freeway segments. A method for analysis of extended lengths of freeway composed of a combination of basic freeway segments, weaving segments, and merge or diverge segments is found in Chapter 10, Freeway Facilities.

Exhibit 11-7 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 11-7
Overview of Operational Analysis Methodology for Basic Freeway Segments

Exhibit 11-7 illustrates the methodology for operational analysis. Other types of analysis are described in the Applications section.



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) demand volume, number and width of lanes, right-side lateral clearance, total ramp density, percent of heavy vehicles (trucks, buses, and RVs), PHF, terrain, and the driver population factor.

Step 2: Compute FFS

FFS can be determined directly from field measurements or can be estimated as described below.

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 1,000 pc/h/ln). For a specific freeway segment, average speeds are virtually constant in this range of flow rates. If the FFS can be field measured, this is the preferable way to make the determination. If the FFS is measured directly, no adjustments are applied to the measured value.

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 tenth 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 Institute of Transportation Engineers *Manual of Traffic Engineering Studies* (*11*).

Estimating FFS

It is not possible to make field measurements for future facilities, and field measurement may not be possible or practical in all existing cases. In such cases, the segment's FFS may be estimated by using Equation 11-1, which is based on the physical characteristics of the segment under study:

Equation 11-1

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84}$$

where

FFS = FFS of basic freeway segment (mi/h),

f_{LW} = adjustment for lane width (mi/h),

f_{LC} = adjustment for right-side lateral clearance (mi/h), and

TRD = total ramp density (ramps/mi).

Base FFS

This methodology covers basic freeway segments with FFSs ranging from 55 mi/h to 75 mi/h. Thus, the predictive algorithm must start with a base speed of 75 mi/h or higher. A value of 75.4 mi/h was chosen, since it resulted in the most accurate predictions versus data collected in 2008.

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 11-8.

Exhibit 11-8
Adjustment to FFS for Average Lane Width

Average Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Adjustment for Lateral Clearance

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 to identify 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, often making their influence on traffic negligible.

Exhibit 11-9 shows the adjustments to the base FFS due to the existence of obstructions closer than 6 ft to the right travel lane edge. Median clearances of 2 ft or more generally have little impact on traffic. No adjustments are available to reflect the presence of left-side lateral obstructions closer than 2 ft to the left travel lane edge. Such situations are, however, quite rare on modern freeways, except in constrained work zones.

Exhibit 11-9
Adjustment to FFS for Right-Side Lateral Clearance, f_{LC} (mi/h)

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

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.

Total Ramp Density

Equation 11-1 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.

Step 3: Select FFS Curve

As noted previously, once the FFS of the basic freeway segment is determined, one of the five base speed–flow curves (Exhibit 11-2) is selected for use in the analysis. Interpolation between curves is not recommended. Criteria for selecting an appropriate curve were given in the text following Exhibit 11-2.

Step 4: Adjust Demand Volume

Since the basic speed–flow curves of Exhibit 11-2 are based on flow rates in equivalent passenger cars per hour, with the driver population dominated by regular users of the basic freeway segment, demand volumes expressed as vehicles per hour under prevailing conditions must be converted to this basis. Equation 11-2 is used for this adjustment:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

Equation 11-2

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,

N = number of lanes in analysis direction,

f_{HV} = adjustment factor for presence of heavy vehicles in traffic stream, and

f_p = adjustment factor for unfamiliar driver populations.

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 11-2 accounts for this phenomenon.

On freeways, typical PHFs range from 0.85 to 0.98 (*13*). 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

A heavy vehicle is defined as any vehicle with more than four wheels on the ground during normal operation. Such vehicles are generally categorized as trucks, buses, or RVs. Trucks cover a wide variety of vehicles, from single-unit trucks with double rear tires to triple-unit tractor–trailer combinations. Small panel or pickup trucks with only four wheels are, however, classified as passenger cars. Buses include intercity buses, public transit buses, and school buses. Because buses are in many ways similar to single-unit trucks, both types of vehicles are considered in one category. RVs include a wide variety of vehicles from self-contained motor homes to cars and small trucks with trailers (for boats, all-terrain vehicles, or other conveyances). It should be noted that most sport-utility vehicles have only four wheels and are thus categorized as passenger cars. The heavy-vehicle adjustment factor f_{HV} is computed as follows:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Equation 11-3

where

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of trucks and buses in traffic stream,

P_R = proportion of RVs in traffic stream,

E_T = passenger-car equivalent (PCE) of one truck or bus in traffic stream, and

E_R = PCE of one RV in traffic stream.

The adjustment factor is found in a two-step process. First, the PCE for each truck or bus and RV 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 or bus or RV under the prevailing conditions. Second, Equation 11-3 is used to convert the PCE values to the adjustment factor.

In many cases, trucks will be the only heavy-vehicle type present in the traffic stream. In others, the percentage of RVs will be small in comparison with trucks and buses. If the ratio of trucks and buses to RVs is 5:1 or greater, all heavy vehicles may be (but do not have to be) considered to be trucks.

The effect of heavy vehicles on traffic flow depends on terrain and grade conditions as well as traffic composition. PCEs can be selected for one of three conditions:

- Extended freeway segments in general terrain,
- Specific upgrades, or
- Specific downgrades.

Each of these conditions is more precisely defined and discussed next.

Equivalents for General Terrain Segments

General terrain refers to extended lengths of freeway 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. As a guideline for this determination, extended segment analysis can be applied where grades are ≤2% and ≤0.25 mi long, or where grades between 2% and 3% are ≤0.50 mi long. For this determination, each upgrade and downgrade is 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. There are three categories of general terrain:

- *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 substantially below those of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length

of time or at frequent intervals. *Crawl speed* is the maximum sustained speed that trucks can maintain on an extended upgrade of a given percent. If the grade is long enough, trucks will be forced to decelerate to the crawl speed, which they can maintain for extended distances. Appendix A contains truck-performance curves illustrating crawl speed and length of grade.

- *Mountainous terrain*: 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.

The mountainous terrain category is rarely used, because individual grades will typically be longer, steeper, or both, than the criteria for general terrain analysis.

Mountainous terrain is relatively rare. Generally, in segments severe enough to cause the type of operation described for mountainous terrain, individual grades will be longer or steeper, or both, than the criteria for general terrain analysis.

Exhibit 11-10 shows PCEs for trucks and buses and RVs in general terrain segments.

Exhibit 11-10
PCEs for Heavy Vehicles in General Terrain Segments

Vehicle	PCE by Type of Terrain		
	Level	Rolling	Mountainous
Trucks and buses, E_T	1.5	2.5	4.5
RVs, E_R	1.2	2.0	4.0

Equivalents for Specific Upgrades

Any freeway grade between 2% and 3% and longer than 0.5 mi or 3% or greater and longer than 0.25 mi should be considered a separate segment. The analysis of such segments must consider the upgrade conditions and the downgrade conditions separately, as well as whether the grade is a single, isolated grade of constant percentage or part of a series forming a composite grade. The analysis of composite grades is discussed in Appendix A.

Several studies have shown that freeway truck populations have an average weight-to-horsepower ratio between 125 and 150 lb/hp. This methodology adopts PCEs that are calibrated for a mix of trucks and buses in this range. RVs vary considerably in both type and characteristics and include everything from cars with trailers to self-contained mobile campers. In addition to the variability of vehicle characteristics, RV drivers are typically not professionals, and their degree of skill in handling such vehicles also varies widely. Typical RV weight-to-horsepower ratios range from 30 to 60 lb/hp.

Exhibit 11-11 and Exhibit 11-12 give values of E_T for trucks and buses and E_R for RVs, respectively. These factors vary with the percent of grade, length of grade, and the proportion of heavy vehicles in the traffic stream. Maximum values occur when there are only a few heavy vehicles in the traffic stream. The equivalents decrease as the number of heavy vehicles increases because these vehicles tend to form platoons. Because heavy vehicles have more uniform operating characteristics, fewer large gaps are created in the traffic stream when they platoon, and the impact of a single heavy vehicle in a platoon is less severe than that of a single heavy vehicle in a stream of primarily passenger cars. The aggregate impact of heavy vehicles on the traffic stream, however, increases as numbers and percentages of heavy vehicles increase.

The grade length should include 25% of the length of the vertical curves at the start and end of the grade.

With two consecutive upgrades, 50% of the length of the vertical curve joining them should be included.

The length of the grade is generally taken from a highway profile. It typically includes the straight portion of the grade plus some portion of the vertical curves at the beginning and end of the grade. It is recommended that 25% of the length of the vertical curves at both ends of the grade be included in the length. Where two consecutive upgrades are present, 50% of the length of the vertical curve joining them is included in the length of each grade.

The point of interest is usually the spot where heavy vehicles would have the greatest impact on operations: the top of a grade, the top of the steepest grade in a series, or a ramp junction, for example.

In the analysis of upgrades, the point of interest is generally at the end of the grade, where heavy vehicles would have the maximum effect on operations. However, if a ramp junction is being analyzed, for example, the length of the grade to the merge or diverge point would be used.

On composite grades, the relative steepness of segments is important. If a 5% upgrade is followed by a 2% upgrade, for example, the maximum impact of heavy vehicles is most likely at the end of the 5% segment. Heavy vehicles would be expected to accelerate after entering the 2% segment.

Exhibit 11-11
PCEs for Trucks and Buses (E_T) on Upgrades

Upgrade (%)	Length (mi)	Proportion of Trucks and Buses 2%	4%	5%	6%	8%	10%	15%	20%	≥25%
≤2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
>2–3	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.50–0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.75–1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>1.00–1.50	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>1.50	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
>3–4	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50–0.75	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	>0.75–1.00	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	>1.00–1.50	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	>1.50	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
>4–5	0.00–0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.50–0.75	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	>0.75–1.00	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	>1.00	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
>5–6	0.00–0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25–0.30	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.30–0.50	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	>0.50–0.75	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	>0.75–1.00	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	>1.00	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
>6	0.00–0.25	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	1.0
	>0.25–0.30	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	>0.30–0.50	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	>0.50–0.75	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	>0.75–1.00	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	>1.00	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

Note: Interpolation for percentage of trucks and buses is recommended to the nearest 0.1.

Exhibit 11-12
PCEs for RVs (E_R) on Upgrades

Upgrade (%)	Length (mi)	Proportion of RVs 2%	4%	5%	6%	8%	10%	15%	20%	≥25%
≤2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
>2–3	0.00–0.50	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.50	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
>3–4	0.00–0.25	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.25–0.50	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
>4–5	0.00–0.25	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>0.25–0.50	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	>0.50	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
>5	0.00–0.25	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	>0.25–0.50	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	>0.50	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0

Note: Interpolation for percentage of RVs is recommended to the nearest 0.1.

Equivalents for Specific Downgrades

Knowledge of specific impacts of heavy vehicles on operating conditions on downgrades is limited. In general, if the downgrade is not severe enough to cause trucks to shift into a lower gear (to engage engine braking), heavy vehicles may be treated as if they were on level terrain segments. Where a downgrade is severe, trucks must often use low gears to avoid gaining too much speed and running out of control. In such cases, their effect on operating conditions is more significant than on level terrain. Exhibit 11-13 gives values of E_T for this situation.

Exhibit 11-13
PCEs for Trucks and Buses (E_T) on Specific Downgrades

Downgrade (%)	Length of Grade (mi)	Proportion of Trucks and Buses 5%	10%	15%	≥20%
<4	All	1.5	1.5	1.5	1.5
4–5	≤4	1.5	1.5	1.5	1.5
	>4	2.0	2.0	2.0	1.5
>5–6	≤4	1.5	1.5	1.5	1.5
	>4	5.5	4.0	4.0	3.0
>6	≤4	1.5	1.5	1.5	1.5
	>4	7.5	6.0	5.5	4.5

On downgrades, RVs are always treated as if they were on level terrain; E_R is therefore always 1.2 on downgrades regardless of the length or severity of the downgrade or the percentage of RVs in the traffic stream.

E_R is always 1.2 on downgrades.

Equivalents for Composite Grades

The vertical alignment of most freeways results in a continuous series of grades. It is often necessary to determine the effect of a series of grades in succession. The most straightforward technique is to compute the average grade from the beginning of the composite grade to the point of interest. The average grade is defined as the total rise from the beginning of the composite grade to the point in question divided by the length of the grade (to the point of interest).

The average-grade technique is an acceptable approach for grades in which all subsections are less than 4% or the total length of the grade is less than 4,000 ft. For more severe composite grades, a detailed technique is presented in Appendix A. This technique uses vehicle performance curves and equivalent speeds to determine the equivalent simple grade for analysis.

The average grade can be used when all component grades are <4% or the total length of the grades is <4,000 ft.

Appendix A provides a method for addressing more severe composite grades.

Adjustment for Driver Population

The base traffic stream characteristics for basic freeway segments are representative of traffic streams composed primarily of commuters, or drivers who are familiar with the facility. It is generally accepted that traffic streams with different characteristics (e.g., recreational drivers) use freeways less efficiently. Although data are sparse and reported results vary substantially, significantly lower capacities have been reported on weekends, particularly in recreational areas. It may generally be assumed that the reduction in capacity (LOS E) extends to service flow rates and service volumes for other LOS as well.

An f_p-value of 1.00 should generally be used, reflective of drivers who are regular users of the freeway.

The adjustment factor f_p is used to reflect the effect of driver population. The values of f_p range from 0.85 to 1.00 in most cases, although lower values have been observed in isolated cases. In general, the analyst should use a value of 1.00, which reflects commuters or otherwise-accustomed drivers, unless there is sufficient evidence that a lower value should be used. Where greater accuracy is needed, comparative field studies of commuter and recreational traffic flow and speeds are recommended.

Does LOS F Exist?

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 segment (2,400 pc/h/ln for FFS = 75 mi/h and 70 mi/h; 2,350 pc/h/ln for FFS = 65 mi/h; 2,300 pc/h/ln for FFS = 60 mi/h; 2,250 pc/h/ln for FFS = 55 mi/h).

If the demand exceeds capacity, the LOS is F, and a breakdown has been identified. To analyze the impacts of such a breakdown, the methodology of Chapter 10, Freeway Facilities, must be used. No further analysis using the methodology of the current chapter is possible.

If the 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 estimated speed and density of the traffic stream may be determined.

With the equations specified in Exhibit 11-3, the expected mean speed of the traffic stream can be computed. A graphical solution with Exhibit 11-2 can also be performed.

With the estimated speed determined, Equation 11-4 is used to estimate the density of the traffic stream:

$$D = \frac{v_p}{S}$$

Equation 11-4

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 11-4 is only used when the 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 11-3 and Exhibit 11-4, and no speed can be estimated.

Where LOS F exists, the analyst is urged to consult Chapter 10, Freeway Facilities, 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 11-5 is entered with the density obtained from Equation 11-4 to determine the expected prevailing LOS.

SENSITIVITY OF RESULTS

The freeway FFS is most sensitive to the total ramp density.

The FFS of basic freeway segments is most sensitive to the total ramp density. Exhibit 11-14 illustrates the resulting FFS when total ramp density varies from 0 ramps/mi to 6 ramps/mi. Standard lane widths and right-side clearances are assumed. A freeway with 0 ramps/mi represents a case in which there are no ramps within 3 mi on either side of the study location. This situation occurs primarily in rural areas, where interchanges may be 10 or more miles apart. In rare cases, ramp densities in excess of 6 ramps/mi may exist, particularly in dense urban areas.

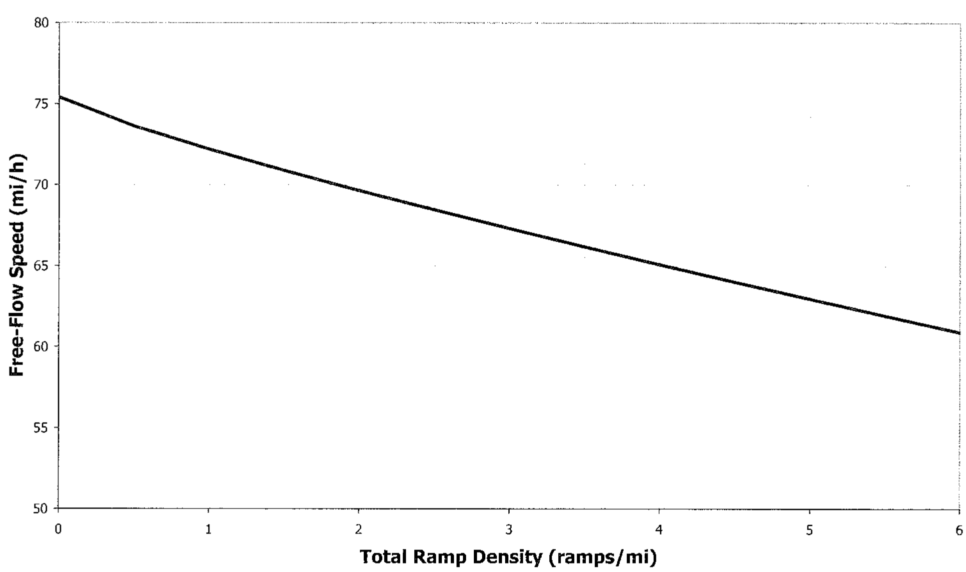


Exhibit 11-14
Sensitivity of FFS to Total Ramp Density

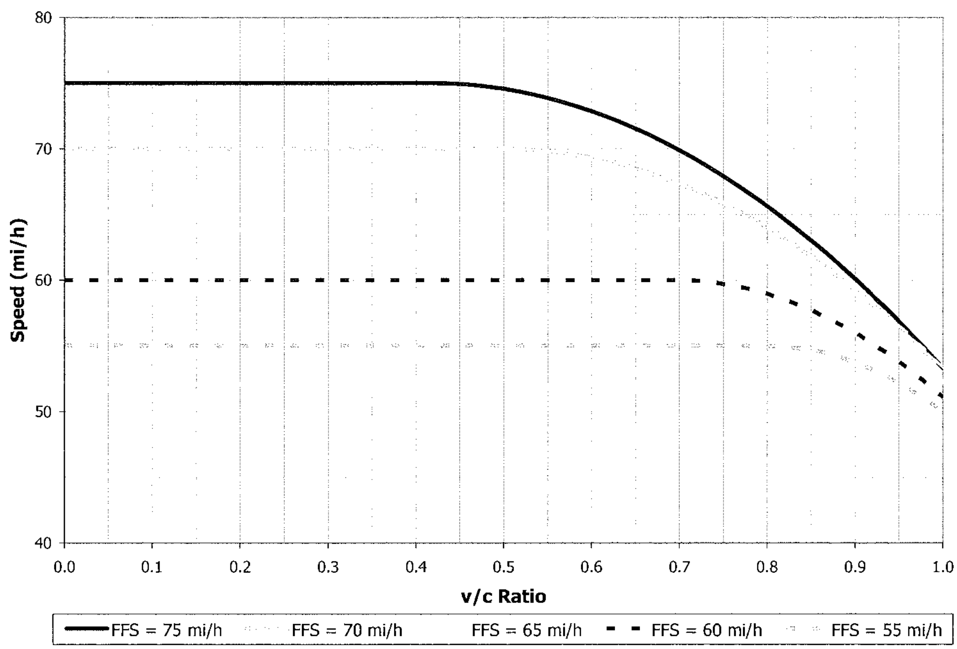
Each on- and off-ramp in the direction of travel is counted when total ramp density is determined.

Higher total ramp densities represent suburban and urban situations as well as the type of interchanges present. 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 finding suggests that in any given situation (with comparable demand flows), cloverleaf interchanges will have a greater negative impact on FFS than diamond interchanges.

Although Exhibit 11-14 is not a straight line, the slope is 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.

Exhibit 11-15 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 11-2. Speed does not begin to decline until a *v/c* ratio of 0.42 to 0.80 is reached, depending on the FFS.

Exhibit 11-15
Speed Versus *v/c* Ratio



3. APPLICATIONS

The methodology in this chapter is relatively straightforward, so it can be directly used in any one of four applications:

1. *Operational analysis*: All traffic and roadway conditions are specified for an existing facility or a future facility with forecast conditions. The existing or expected LOS is determined.
2. *Design analysis*: A forecast demand volume is used, and key design parameters are specified (e.g., lane width and lateral clearance). The number of lanes required to deliver a target LOS is determined.
3. *Planning and preliminary engineering*: The basic scenario is the same as that for design analysis, except that the analysis is conducted at a much earlier stage of the development process. Inputs include default values, and the demand volume is usually stated as an annual average daily traffic (AADT) value.
4. *Service flow rates and service volumes*: The service flow rate, service volume, or daily service volume, or all three, are estimated for each LOS for an existing or future facility. All traffic and roadway conditions must be specified for this type of analysis.

Because the methodology and its algorithms are simple and do not involve iterations, all of the types of analysis cited can be done without the trial-and-error approach required by many other *Highway Capacity Manual* (HCM) methodologies.

DEFAULT VALUES

In using this chapter's methodology, a range of input data is needed. Most of these data should be field-measured or estimated values for the specific segment under consideration. When some of the data are not available, default values may be used. However, the use of default values will affect the accuracy of the output. Exhibit 11-16 shows the data that are required to conduct an operational analysis and the recommended default values when site-specific data are unavailable (*13*).

Exhibit 11-16
Required Input Data and Default Values for Basic Freeway Segments

Required Data	Default Values
Geometric Data	
Number of lanes in one direction	No default, must have site-specific value
Lane width (ft)	12 ft
Right-side lateral clearance (ft)	10 ft
Ramp density (ramps/mi)	No default, must have site-specific value
Terrain or specific grade (%, length)	No default, must have site-specific value
FFS (mi/h)	Urban, 70 mi/h; rural, 75 mi/h
Demand Data	
Length of analysis period (min)	15 min
PHF	0.94
Proportion of heavy vehicles (%)	Urban, 5%; rural, 12%*
Driver population factor	1.00

* Alternative state-specific default values for percentage of heavy vehicles are given in Chapter 26.

The analyst may also replace the default values of Exhibit 11-16 with defaults that have been locally calibrated.

Research into the percentage of heavy vehicles on uninterrupted-flow facilities (*13*) found a wide range of average values from state to state. Chapter 26 provides alternative default values for percentage of heavy vehicles by state and area population on the basis of data from the 2004 Highway Performance Monitoring System. Where states or local jurisdictions have developed their own values, these may be substituted. Analysts may also wish to develop their own default values based on more recent data.

ESTABLISH ANALYSIS BOUNDARIES

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 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 segments.

Ramp junctions, grade changes of 2% or more, changes in the freeway's geometric characteristics, and changes in speed limit are places where basic freeway segment boundaries should be established.

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

- 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), or
- Change in posted speed limit.

The last is not directly involved in the analysis of a basic freeway segment but would probably reflect changes in ramp density or other freeway features.

TYPES OF ANALYSIS

Operational Analysis

Operational analyses find the expected LOS for specified roadway and traffic conditions.

The operational analysis application was fully specified in the Methodology section of this chapter. Operational analysis begins with all input parameters specified and is used to find the expected LOS that would result from the prevailing roadway and traffic conditions.

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, using Equation 11-5 instead of Equation 11-2:

Equation 11-5

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

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 11-17. These values are selected from the base speed–flow curves of Exhibit 11-6 for each LOS.

Exhibit 11-17
Maximum Service Flow Rates in Passenger Cars per Hour per Lane for Basic Freeway Segments Under Base Conditions

FFS	Target Level of Service				
(mi/h)	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.

Next the number of lanes required to deliver the target LOS can be found from Equation 11-6:

Equation 11-6

$$N = \frac{v}{MSF_i}$$

where N is the number of lanes required and MSF_i is the maximum service flow rate for LOS i from Exhibit 11-17. Equation 11-5 and Equation 11-6 can be conveniently combined as Equation 11-7:

Equation 11-7

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

where all variables are as previously defined.

All fractional values of N must be rounded up.

The value of N resulting from Equation 11-6 or Equation 11-7 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, it may not be possible to achieve the target LOS for a given demand volume.

This rounding-up process will occasionally produce an interesting result: it is possible that a target LOS (for example, LOS C) cannot be achieved 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 actually improve to B. Thus, 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 slightly lower operating condition than that originally targeted.

Planning and Preliminary Engineering

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.

The objective of planning or preliminary engineering is to get 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 be usually 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 11-8:

Equation 11-8

$$V = DDHV = AADT \times K \times D$$

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; all other variables are as previously defined.

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 was 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.

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 a basic freeway segment.

Exhibit 11-17 gave values of the maximum service flow rates, MSF_i, for each LOS for freeways of various FFSs. 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 11-9:

Equation 11-9

$$SF_i = MSF_i \times N \times f_{HV} \times f_p$$

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 11-10. 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 11-10

$$SV_i = SF_i \times PHF$$

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 11-11:

Equation 11-11

$$DSV_i = \frac{SV_i}{K \times D}$$

where all variables are as previously defined.

Service flow rates *SF* and service volumes *SV* are stated for a single direction of the freeway. Daily service volumes *DSV* are stated as total volumes in *both* directions of the freeway.

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 segments. Additional information on this topic may be found in Chapter 26, Basic Freeway Segments: Supplemental.

Strengths of HCM Procedure

The HCM methodology provides FFS as an output, incorporates geometric characteristics, provides explicit capacity estimates, and produces a single deterministic estimate of traffic density.

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 a body of expert consensus.

Specific strengths of the HCM basic freeway segment methodology include the following:

- It provides a detailed methodology for obtaining FFS. This methodology is based on various geometric characteristics. In simulation packages FFS (or an equivalent, such as desired speed) is an input.
- It considers geometric characteristics (such as lane widths), which are rarely, if ever, incorporated into simulation algorithms.
- It provides explicit capacity estimates. Simulation packages do not provide capacity estimates directly. Capacity estimates can only be obtained from simulators through multiple runs with oversaturated conditions. The user can modify simulated capacities by modifying specific input values such as the minimum acceptable headway.
- It produces a single deterministic estimate of traffic density, which is important for some purposes such as development impact review.

Limitations of HCM Procedures That Might Be Addressed by Alternative Tools

Deterministic models yield the same results for the same inputs each time they are implemented; stochastic models incorporate statistical variability. The same inputs yield different results in each use. For such models, an average result of X usages is employed as output.

Basic freeway segments can be analyzed by using a variety of stochastic and deterministic simulation packages that include freeways. These packages can be very 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 11-18 tabulates the HCM limitations for basic freeway segments along with the potential for improved treatment by alternative tools.

Additional Features and Performance Measures Available from Alternative Tools

This chapter provides a methodology for estimating the capacity, speed, and density of a basic freeway segment, given the segment's traffic demand and characteristics. Alternative tools offer additional performance measures, including delay, stops, queue lengths, fuel consumption, pollution, and operating costs.

Exhibit 11-18
Limitations of HCM Basic Freeway Segments Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Special lanes reserved for a single vehicle type, such as HOV, truck, and climbing lanes	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	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 given segment
Presence of ITS features related to vehicle or driver guidance	Several features modeled explicitly by simulation; others may be approximated by using assumptions (for example, by modifying origin–destination demands by time interval)

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 a situation 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 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, but because there is no variability in the speeds, the space mean speed is equal to the time 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, it is important to take into account the following:

- 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.

Regarding capacity, 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 these 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's 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 broad range of flows. However, this is not necessarily the case for any given simulation package, some of which assume a continuously decreasing speed with increasing flow. Furthermore, in some simulation packages, that relationship changes when certain parameters are modified. Therefore, if performance measures are compatible between the HCM and an alternative model for a given set of flows, this will not necessarily be the case for all other sets of flows.

Adjustment of Simulation Parameters to HCM Results

The most important elements to be adjusted when a basic freeway segment is analyzed are the speed–flow relationship or 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. Some tools only accept integer values of FFS, whereas the HCM may provide a fractional value as an intermediate calculation result.

Step-by-Step Recommendations for Applying Alternative Tools

This section provides recommendations specifically for freeway 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 segments, the following steps should be taken:

1. Determine whether the chosen tool can provide density and capacity for a basic freeway 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 required number of runs to be conducted so that the comparison is statistically valid.
5. If the analysis requires evaluating various different 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. It is possible that the simulation cannot be calibrated to match the HCM speed–flow relationship. 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, 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 are based on Example Problem 3 (found in the next section of this chapter), which analyzes 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.

4. EXAMPLE PROBLEMS

Exhibit 11-19
List of Example Problems

Example Problem	Description	Application
1	Four-lane freeway LOS	Operational analysis
2	Number of lanes required for target LOS	Design analysis
3	Six-lane freeway LOS and capacity	Operational and planning analysis
4	LOS on upgrades and downgrades	Operational analysis
5	Design-hour volume and number of lanes	Planning analysis
6	Service flow rates and service volumes	Planning analysis

EXAMPLE PROBLEM 1: FOUR-LANE FREEWAY LOS

The Facts

- Four-lane freeway (two lanes in each direction);
- Lane width = 11 ft;
- Right-side lateral clearance = 2 ft;
- Commuter traffic (regular users);
- Peak-hour, peak-direction demand volume = 2,000 veh/h;
- Traffic composition: 5% trucks, 0% RVs;
- PHF = 0.92;
- One cloverleaf interchange per mile; and
- Rolling terrain.

Comments

The task is to find the expected LOS for this freeway during the worst 15 min of the peak hour. With one cloverleaf interchange per mile, the total ramp density will be 4 ramps/mi.

Step 1: Input Data

All input data are specified above.

Step 2: Compute FFS

The FFS of the freeway is estimated as follows:

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22\,TRD^{0.84}$$

The adjustment for lane width is selected from Exhibit 11-8 for 11-ft lanes (1.9 mi/h). The adjustment for right-side lateral clearance is selected from Exhibit 11-9 for a 2-ft clearance on a freeway with two lanes in one direction (2.4 mi/h). The total ramp density is 4 ramps/mi. Then

$$FFS = 75.4 - 1.9 - 2.4 - 3.22\left(4^{0.84}\right) = 60.8 \text{ mi/h}$$

Step 3: Select FFS Curve

As the FFS calculated in Step 2 is greater than or equal to 57.5 and less than 62.5 mi/h, the 60-mi/h speed–flow curve will be used for this analysis.

Step 4: Adjust Demand Volume

The demand volume must be adjusted to a flow rate that reflects passenger cars per hour per lane under equivalent base conditions by using Equation 11-2:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

The demand volume is given as 2,000 veh/h. The PHF is specified to be 0.92, and there are two lanes in each direction. The driver population factor is 1.00, since regular users (commuters) are also specified. Trucks make up 5% of the traffic stream, so a heavy-vehicle adjustment factor must be determined.

From Exhibit 11-10, the PCE for trucks is 2.5 for rolling terrain. The heavy-vehicle adjustment factor is then computed by using Equation 11-3:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1) + 0} = 0.930$$

Then

$$v_p = \frac{2{,}000}{0.92 \times 2 \times 0.93 \times 1.00} = 1{,}169 \text{ pc/h/ln}$$

Since this value is less than the base capacity of 2,300 pc/h/ln for a freeway with FFS = 60 mi/h, LOS F does not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

The FFS of the basic freeway segment is now estimated along with the demand flow rate in passenger cars per hour per lane under equivalent base conditions. From Exhibit 11-3, the equation for estimating the speed of the traffic stream is selected for a 60-mi/h FFS, with a flow rate less than 1,600 pc/h/ln. This is the constant-speed portion of the curve, so S = 60 mi/h. The density of the traffic stream may now be computed as

$$D = \frac{v_p}{S} = \frac{1{,}169}{60} = 19.5 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 11-5, a density of 19.5 pc/mi/ln corresponds to LOS C but is close to the boundary for LOS B, which is a maximum of 18 pc/mi/ln. This solution could also be calculated graphically by using Exhibit 11-6 as a base (Exhibit 11-20).

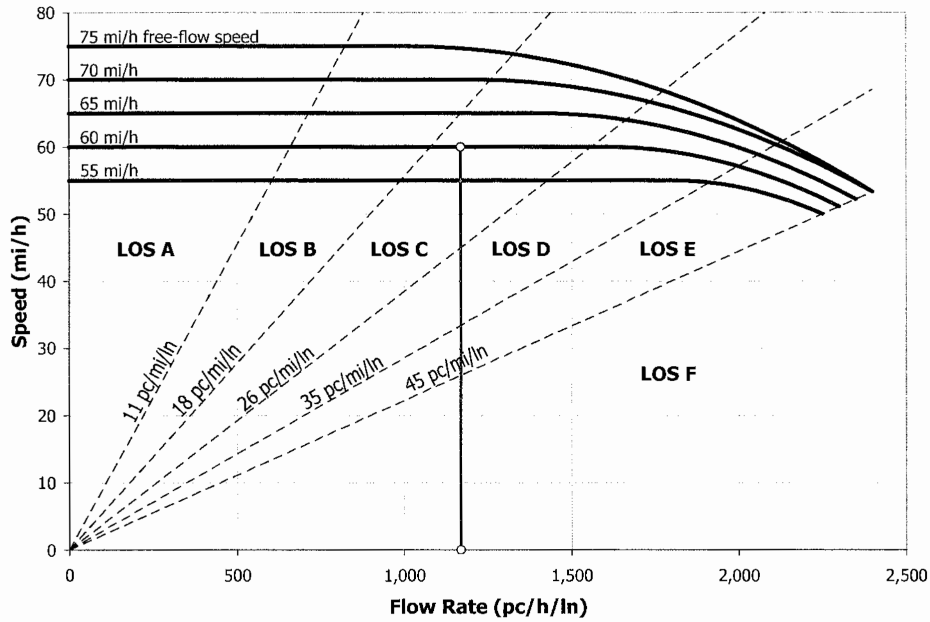


Exhibit 11-20
Graphical Solution for Example Problem 1

Discussion

This basic freeway segment of a four-lane freeway is expected to operate at LOS C during the worst 15 min of the peak hour. It is important to note that the operation, although at LOS C, is close to the LOS B boundary. In most jurisdictions, this operation would be considered to be quite acceptable; therefore, no remediation would normally be required.

EXAMPLE PROBLEM 2: NUMBER OF LANES REQUIRED FOR TARGET LOS

The Facts

- Demand volume = 4,000 veh/h (one direction);
- Level terrain;
- Traffic composition: 15% trucks, 3% RVs;
- Provision of 12-ft lanes;
- Provision of 6-ft right-side lateral clearance;
- Commuter traffic (regular users);
- PHF = 0.85;
- Ramp density = 3 ramps/mi; and
- Target LOS = D.

Comments

This is a classic design application of the methodology. The number of lanes needed to provide LOS D during the worst 15 min of the peak hour is to be determined.

Step 1: Input Data

All input data were specified previously.

Step 2: Compute FFS

The FFS is estimated by using Equation 11-1. Because the lane width and lateral clearance to be provided on the new freeway will be 12 ft and 6 ft, respectively, there are no adjustments for these features. The total ramp density is given as 3 ramps/mi. Then

$$FFS = 75.4 - f_{LW} - f_{LC} - 3.22TRD^{0.84}$$

$$FFS = 75.4 - 0.0 - 0.0 - 3.22\left(3^{0.84}\right) = 67.3 \text{ mi/h}$$

Step 3: Select FFS Curve

Since the FFS calculated in Step 2 is greater than or equal to 62.5 and less than 67.5 mi/h, the 65-mi/h speed–flow curve will be used for this analysis.

Step 4: Estimate Number of Lanes Needed

Because this is a design analysis, Step 4 of the operational analysis methodology is modified. Equation 11-7 may be used directly to determine the number of lanes needed to provide for at least LOS D:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

A value of the maximum service flow rate must be selected from Exhibit 11-17 for a FFS of 65 mi/h and LOS D. This value is 2,030 pc/h/ln. The PHF is given as 0.85. The driver population factor is 1.00, since commuters are involved. A heavy-vehicle factor for 15% trucks and 3% RVs must be determined by using Exhibit 11-10 for level terrain. The PCEs of trucks and RVs in level terrain are 1.5 and 1.2, respectively. Then

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right) + P_R\left(E_R - 1\right)}$$

$$f_{HV} = \frac{1}{1 + 0.15(1.5 - 1) + 0.03(1.2 - 1)} = 0.925$$

and

$$N = \frac{4{,}000}{2030 \times 0.85 \times 0.925 \times 1.00} = 2.51 \text{ lanes}$$

It is not possible to build 2.51 lanes. To provide a minimum of LOS D, it will be necessary to provide three lanes in each direction, or a six-lane freeway.

At this point, the design application ends. It is possible, however, to consider what speed, density, and LOS will prevail when three lanes are actually provided. Therefore, the example problem continues with Steps 5 and 6.

Step 5: Estimate Speed and Density

In pursuing additional information, the problem now reverts to an operational analysis of a three-lane basic freeway segment with a demand volume of 4,000 pc/h.

Equation 11-2 is used to compute the actual demand flow rate per lane under equivalent base conditions:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

$$v_p = \frac{4{,}000}{0.85 \times 3 \times 0.925 \times 1.00} = 1{,}696 \text{ pc/h/ln}$$

The expected speed of the traffic stream may be estimated either by using Exhibit 11-6 (for a graphical solution) or by selecting the appropriate equation from Exhibit 11-3—in this case, using FFS = 65 mi/h and a demand flow rate over 1,400 pc/h/ln. With the latter approach,

$$S = 65 - 0.00001418\left(v_p - 1{,}400\right)^2$$

$$S = 65 - 0.00001418\,(1{,}696 - 1{,}400)^2 = 63.8 \text{ mi/h}$$

The density may now be computed:

$$D = \frac{v_p}{S} = \frac{1{,}696}{63.8} = 26.6 \text{ pc/mi/ln}$$

Step 6: Determine LOS

Entering Exhibit 11-5 with a density of 26.6 pc/mi/ln, the LOS is D but is very close to the boundary of LOS C, which is 26 pc/mi/ln.

Discussion

The resulting LOS is D, which was the target for the design. Although the minimum number of lanes needed was 2.51, which would have provided for a minimal LOS D, providing three lanes yields a density that is close to the LOS C boundary. In any event, the target LOS of the design will be met by providing a six-lane basic freeway segment.

EXAMPLE PROBLEM 3: SIX-LANE FREEWAY LOS AND CAPACITY

The Facts

- Volume of 5,000 veh/h (one direction, existing);
- Volume of 5,600 veh/h (one direction, in 3 years);
- Traffic composition: 10% trucks, no RVs;
- Level terrain;
- Three lanes in each direction;
- FFS = 70 mi/h (measured);
- PHF = 0.95;
- Commuter traffic (regular users); and
- Traffic growth after 3 years = 4% per year.

Comments

This example consists of two operational analyses, one for the present demand volume of 5,000 pc/h and one for the demand volume of 5,600 pc/h expected in 3 years. In addition, a planning element is introduced: Assuming that traffic grows as expected, when will the capacity of the roadway be exceeded? This analysis requires that capacity be determined in addition to the normal output of operational analyses.

Step 1: Input Data

All input data were given previously.

Step 2: Compute FFS

Step 2 is not needed since a measured FFS is given (70 mi/h).

Step 3: Select FFS Curve

Step 3 is not needed. The FFS curve for 70 mi/h will be used, based on the measured value.

Step 4: Adjust Demand Volume

In this case, two demand volumes will be adjusted by using Equation 11-2:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

The PHF is given as 0.95, and there are three lanes in each direction. The driver population adjustment factor will be 1.00, for regular users. The heavy-vehicle factor must reflect 10% trucks in level terrain. From Exhibit 11-10, the PCE for trucks in level terrain is 1.5. Equation 11-3 then gives the following:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.10(1.5 - 1) + 0} = 0.952$$

Two values of v_p will be computed: one for present conditions and one for conditions in 3 years:

$$v_p\text{ (present)} = \frac{5{,}000}{0.95 \times 3 \times 0.952 \times 1.00} = 1{,}843\text{ pc/h}$$

$$v_p\text{ (future)} = \frac{5{,}600}{0.95 \times 3 \times 0.952 \times 1.00} = 2{,}064\text{ pc/h}$$

Step 5: Estimate Speed and Density

Two values of speed and density will be estimated, one each for the present and future conditions stated. The equations of Exhibit 11-3 will be used to estimate speeds. One equation applies to both cases, a 70-mi/h FFS with a flow rate over 1,200 pc/h/ln:

$$S(\text{present}) = 70 - 0.00001160(v_p - 1{,}200)^2$$

$$S(\text{present}) = 70 - 0.00001160(1{,}843 - 1{,}200)^2 = 65.2 \text{ mi/h}$$

$$S(\text{future}) = 70 - 0.00001160(v_p - 1{,}200)^2$$

$$S(\text{future}) = 70 - 0.00001160(2{,}064 - 1{,}200)^2 = 61.3 \text{ mi/h}$$

The corresponding densities may now be estimated as follows:

$$D = \frac{v_p}{S}$$

$$D(\text{present}) = \frac{1{,}843}{65.2} = 28.3 \text{ pc/mi/ln}$$

$$D(\text{future}) = \frac{2{,}064}{61.3} = 33.7 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 11-5, the LOS for the present situation is D, and the LOS for the future scenario (in 3 years) is also D, despite the increase in density.

Step 7: When Will Capacity Be Reached?

Step 7 is an additional step for this problem. To answer the question, the capacity of the basic freeway segment must be estimated. From Exhibit 11-17, the maximum service flow rate for LOS E on a basic freeway segment with a 70-mi/h FFS is 2,400 pc/h/ln. This flow rate is synonymous with capacity.

The analyst must be sure that the capacity and demand flow rates compared in Step 7 are on the same basis. The 2,400 pc/h/ln is a flow rate under equivalent base conditions. The demand flow rate in 3 years was estimated to be 2,064 pc/h/ln on this basis. These two values, therefore, may be compared. As an alternative, the capacity could be computed for prevailing conditions:

$$SF_E = MSF_E \times N \times f_{HV} \times f_p$$

$$SF_E = 2{,}400 \times 3 \times 0.952 \times 1.00 = 6{,}854 \text{ veh/h}$$

This capacity, however, is stated as a *flow rate*. The demand volume is stated as an hourly volume. Thus, a *service volume* for LOS E is needed:

$$SV_E = SF_E \times PHF = 6{,}854 \times 0.95 = 6{,}511 \text{ veh/h}$$

The problem may be solved either by comparing the demand volume of 5,600 veh/h (in 3 years) with the hourly capacity of 6,511 veh/h or by comparing the demand flow rate under equivalent base conditions of 2,064 pc/h/ln with the base capacity of 2,400 pc/h/ln. With the hourly demand volume and hourly capacity,

$$6{,}511 = 5{,}600(1.04)^n$$

$$n = 3.85 \text{ years}$$

On the basis of the forecasts of traffic growth, the basic freeway segment described will reach capacity within 7 years (the demand of 5,600 veh/h occurs 3 years from the present).

Discussion

The LOS on this segment will remain D within 3 years despite the increase in density. The demand is expected to exceed capacity within 7 years. Given the normal lead times for planning, design, and approvals before the start of construction, it is probable that planning and preliminary design for an improvement should be started immediately.

EXAMPLE PROBLEM 4: LOS ON UPGRADES AND DOWNGRADES

The Facts

- Demand volume = 2,300 veh/h (one direction);
- Traffic composition: 15% trucks, no RVs;
- PHF = 0.90;
- FFS = 70 mi/h upgrade, 75 mi/h downgrade (measured);
- Unfamiliar drivers (f_p = 0.95); and
- Composite grade: 3,000 ft at 3%, followed by 2,600 ft at 5%.

Comments

This is a typical operational analysis. The expected outcome is an assessment of the LOS on both the upgrade and the downgrade. However, the problem deals with a specific grade and a composite grade. Because there is a segment of the grade that is greater than 4% and the total length of the composite grade exceeds 4,000 ft, the special procedure in Appendix A must be applied. That procedure will yield an equivalent constant-percent grade of 3,000 + 2,600 = 5,600 ft (1.06 mi), which has the same impact on heavy vehicles as the composite grade described.

Composite Grade

Exhibit 11-21 shows the conversion of the composite grade to a grade of constant percent 5,600 ft long. At the end of such a grade, the final speed of heavy vehicles is approximately the same as that on the composite grade.

A vertical line enters the truck performance curves at 3,000 ft extending to the +3% grade curve, indicating that the speed of trucks after 3,000 ft of +3% grade is approximately 42 mi/h. This is also the speed at which the truck enters the +5% grade; it corresponds to the same speed as that of a truck on a +5% grade after 1,300 ft. The truck travels another 2,600 ft (to 3,900 ft) on the +5% curve, where a final speed of 27 mi/h is reached. The intersection of a horizontal drawn at 27 mi/h and a vertical drawn at a total length of grade of 5,600 ft yields the equivalent of +5%. In effect, because trucks on this grade are at crawl speed, it does not matter how long the grade is: 27 mi/h can be maintained indefinitely.

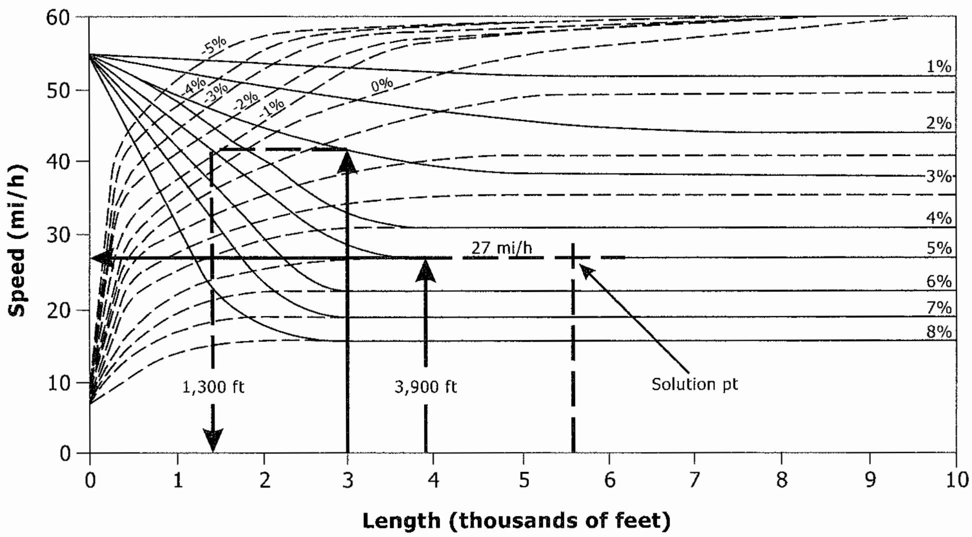


Exhibit 11-21
Determination of Composite Grade Equivalents for Example Problem 4

The equivalent grade is 5%, 5,600 ft. This equivalent should be applied to *both the upgrade and the downgrade*, even though it is developed specifically for the upgrade.

Although the truck acceleration curves of Appendix A could be used to develop a separate downgrade composite equivalent, it would be very misleading. The truck performance curves assume a maximum speed of 60 mi/h. On a long, steep downgrade, trucks will achieve much higher speeds.

It is highly likely that trucks will be forced to use a low gear to apply engine braking on the grade described. Thus, PCEs for the downgrade will be selected from Exhibit 11-13.

Step 1: Input Data

All input data were specified previously.

Step 2: Compute FFS

FFSs were measured in the field. The upgrade FFS is 70 mi/h; the downgrade FFS is 75 mi/h.

Step 3: Select FFS Curve

The 70-mi/h curve will be used for the upgrade; the 75-mi/h curve will be used for the downgrade.

Step 4: Adjust Demand Volume

The demand flow rates in passenger cars per hour per lane for the upgrade and downgrade are estimated by using Equation 11-2:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

The PHF is 0.90, there are two lanes on the upgrade and two lanes on the downgrade, and f_p is specified as 0.95. Heavy-vehicle adjustment factors, however, must be determined separately for the upgrade and the downgrade.

The PCE for trucks (E_T) on the upgrade is selected from Exhibit 11-11 for a grade of 5%, >1.00 mi long, with 15% trucks: 3.0. The PCE for the trucks on the downgrade is selected from Exhibit 11-13 for a grade of 4% to 5%, ≤4 mi long: 1.5.

The heavy-vehicle adjustment factors, f_{HV}, are computed by using Equation 11-3:

$$f_{HV} = \frac{1}{1 + P_T\left(E_T - 1\right) + P_R\left(E_R - 1\right)}$$

$$f_{HV}\left(\text{upgrade}\right) = \frac{1}{1 + 0.15(3-1) + 0} = 0.769$$

$$f_{HV}\left(\text{downgrade}\right) = \frac{1}{1 + 0.15(1.5-1) + 0} = 0.930$$

Then

$$v_p\left(\text{upgrade}\right) = \frac{2{,}300}{0.90 \times 2 \times 0.769 \times 0.95} = 1{,}749 \text{ pc/h/ln}$$

$$v_p\left(\text{downgrade}\right) = \frac{2{,}300}{0.90 \times 2 \times 0.930 \times 0.95} = 1{,}446 \text{ pc/h/ln}$$

Since neither of these values exceeds the base capacity of a freeway with FFS = 75 mi/h (downgrade) or FFS = 70 mi/h (upgrade), LOS F does not exist, and the analysis continues to Step 5.

Step 5: Estimate Speed and Density

With the FFS and the demand flow rate determined for both the upgrade and the downgrade, the expected speed and density on each may now be estimated. Speed is estimated by using the equations of Exhibit 11-3.

For the upgrade, the FFS is 70 mi/h, and the demand flow rate is greater than 1,200 pc/h/ln. Then

$$S = 70 - 0.00001160\left(v_p - 1{,}200\right)^2$$

$$S = 70 - 0.00001160\left(1{,}749 - 1{,}200\right)^2 = 66.5 \text{ mi/h}$$

For the downgrade, the FFS is 75 mi/h, and the demand flow rate is greater than 1,000 pc/h/ln. Then

$$S = 75 - 0.00001107\left(v_p - 1{,}000\right)^2$$

$$S = 75 - 0.00001107\left(1{,}446 - 1{,}000\right)^2 = 72.8 \text{ mi/h}$$

Densities may now be estimated from the demand flow rates and estimated speeds:

$$D = \frac{v_p}{S}$$

$$D(\text{upgrade}) = \frac{1{,}749}{66.5} = 26.3 \text{ pc/mi/ln}$$

$$D(\text{downgrade}) = \frac{1{,}446}{72.8} = 19.9 \text{ pc/mi/ln}$$

Step 6: Determine LOS

As shown in Exhibit 11-5, the upgrade LOS is D; the downgrade LOS is C. Both levels, however, are close to the boundaries for better operations—the upgrade is close to the boundary for LOS C (D = 26 pc/mi/ln) and the downgrade is close to the boundary for LOS B (D = 18 pc/mi/ln).

Discussion

Both the upgrade and the downgrade are operating at what would generally be called acceptable levels. If traffic grows over time, the addition of a truck climbing lane on the upgrade might be considered.

EXAMPLE PROBLEM 5: DESIGN-HOUR VOLUME AND NUMBER OF LANES

The Facts

- Demand volume = 75,000 veh/day,
- Proportion of AADT in the peak hour: 0.09,
- Directional distribution: 55/45,
- Rolling terrain, and
- Target LOS = D.

Comments

In this planning and preliminary engineering application, several input variables are not specified, so default values will have to be used. With knowledge of local conditions and freeway design standards, the following default values will be used in the solution: FFS = 65 mi/h; 5% trucks, no RVs; PHF = 0.95; and f_p = 1.00.

Determining Opening-Day Directional Design-Hour Volume

Because the demand volume is given as an AADT, it must be converted to a directional design-hour volume (DDHV) by using Equation 11-8:

$$V = DDHV = AADT \times K \times D$$

$$V = DDHV = 75{,}000 \times 0.09 \times 0.55 = 3{,}713 \text{ veh/h}$$

Step 1: Input Data

All input data were specified.

Step 2: Compute FFS

A default value of 65 mi/h will be used in this problem.

Step 3: Select FFS Curve

The 65-mi/h speed–flow curve will be used in this problem.

Step 4: Determine Number of Lanes Required

After estimating the demand volume on an hourly basis, the remainder of this solution follows the design application. The number of lanes needed is estimated by using Equation 11-7:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

The maximum service flow rate is selected from Exhibit 11-17 for LOS D on a 65-mi/h basic freeway segment: 2,030 pc/h/ln. The PHF is a default value: 0.95. The driver population factor is also a default value: 1.00. The freeway is in rolling terrain and is expected to have 5% trucks (another default value). From Equation 11-10, for rolling terrain, $E_T = 2.5$. Then

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1) + 0} = 0.930$$

$$N = \frac{3{,}713}{2{,}030 \times 0.95 \times 0.93 \times 1.00} = 2.07 \text{ lanes}$$

Because fractional lanes cannot be built, three lanes will have to be provided in each direction to ensure that LOS D is provided during the worst 15 min of the peak hour. Therefore, the resulting LOS may be better than the design target.

Step 5: Estimate Speed and Density

In order to determine the likely LOS resulting from a six-lane freeway, the speed and density should be estimated. Equation 11-2 is used to determine the actual demand flow rate for three lanes:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

$$v_p = \frac{3{,}713}{0.95 \times 3 \times 0.93 \times 1.00} = 1{,}401 \text{ pc/h/ln}$$

From Exhibit 11-3, for a 65-mi/h basic freeway segment with more than 1,400 pc/h/ln, the expected speed is

$$S = 65 - 0.00001418\left(v_p - 1{,}400\right)^2$$

$$S = 65 - 0.00001418(1{,}401 - 1{,}400)^2 = 65.0 \text{ mi/h}$$

and the density is

$$D = \frac{v_p}{S} = \frac{1{,}401}{65.0} = 21.6 \text{ pc/mi/ln}$$

Step 6: Determine LOS

As shown in Exhibit 11-5, the expected LOS is C.

Discussion

This problem illustrates an interesting point: given the parameters of this example problem, the target LOS of D cannot be achieved on opening day. If a four-lane freeway (two lanes in each direction) is built, LOS E will result. If a six-lane freeway (three lanes in each direction) is built, LOS C will result.

EXAMPLE PROBLEM 6: SERVICE FLOW RATES AND SERVICE VOLUMES

The Facts

- Eight-lane freeway;
- FFS = 70 mi/h (measured);
- Traffic composition: 8% trucks, 1% RVs;
- Rolling terrain;
- PHF = 0.87;
- Driver population factor f_p = 1.00;
- Proportion of AADT in peak hour (*K*-factor): 0.08; and
- Directional distribution (*D*-factor): 60/40.

Comments

In this problem, the service flow rate, service volume, and daily service volume for each LOS will be computed. These values could then be compared with any existing or forecast demand volumes to determine the LOS.

Step 1: Input Data

All input data were specified.

Step 2: Compute FFS

The FFS has been field-measured as 70 mi/h.

Step 3: Select FFS Curve

The curve for FFS = 70 mi/h will be used.

Step 4: Compute Service Flow Rates, *SF*

For a 70-mi/h basic freeway segment, maximum service flow rates, *MSF*, can be selected from Exhibit 11-17. These are the maximum service flow rates that can be sustained while a given LOS is maintained. They are stated as flow rates in passenger cars per hour per lane for equivalent base conditions. The values are

- MSF_A = 770 pc/h/ln,
- MSF_B = 1,250 pc/h/ln,
- MSF_C = 1,690 pc/h/ln,
- MSF_D = 2,080 pc/h/ln, and
- MSF_E = 2,400 pc/h/ln.

Service flow rates, *SF*, are estimated by using Equation 11-9:

$$SF_i = MSF_i \times N \times f_{HV} \times f_p$$

where the maximum service flow rates are as cited, $N = 4$ lanes in each direction, and the driver population factor f_p is 1.00. The heavy-vehicle adjustment factor must be determined for 8% trucks and 1% RVs in rolling terrain. From Exhibit 11-10, for rolling terrain, $E_T = 2.5$ and $E_R = 2.0$. Then

$$f_{HV} = \frac{1}{1+0.08(2.5-1)+0.01(2.0-1)} = 0.885$$

Service flow rates may now be computed:

$$SF_A = 770 \times 4 \times 0.885 \times 1.00 = 2{,}726 \text{ veh/h}$$

$$SF_B = 1{,}250 \times 4 \times 0.885 \times 1.00 = 4{,}425 \text{ veh/h}$$

$$SF_C = 1{,}690 \times 4 \times 0.885 \times 1.00 = 5{,}983 \text{ veh/h}$$

$$SF_D = 2{,}080 \times 4 \times 0.885 \times 1.00 = 7{,}363 \text{ veh/h}$$

$$SF_E = 2{,}400 \times 4 \times 0.885 \times 1.00 = 8{,}496 \text{ veh/h}$$

Service flow rates are the maximum rates of flow that may exist in the worst 15-min period of the peak hour while the stated LOS is maintained.

Step 5: Compute Service Volumes, *SV*

Equation 11-10 is used to convert service flow rates to service volumes. The conversion multiplies the service flow rates by the PHF to produce maximum hourly volumes that can be accommodated while the given LOS is maintained during the worst 15 min of the hour.

$$SV_i = SF_i \times PHF$$

$$SV_A = 2{,}726 \times 0.87 = 2{,}372 \text{ veh/h}$$

$$SV_B = 4{,}425 \times 0.87 = 3{,}850 \text{ veh/h}$$

$$SV_C = 5{,}983 \times 0.87 = 5{,}205 \text{ veh/h}$$

$$SV_D = 7{,}363 \times 0.87 = 6{,}406 \text{ veh/h}$$

$$SV_E = 8{,}496 \times 0.87 = 7{,}392 \text{ veh/h}$$

Step 6: Compute Daily Service Volumes, *DSV*

Equation 11-11 is used to convert service volumes to daily service volumes. Daily service volumes are the maximum AADTs that can be accommodated while the given LOS is maintained during the worst 15 min of the peak hour in the peak direction of flow.

$$DSV_i = \frac{SV_i}{K \times D}$$

$$DSV_A = \frac{2{,}372}{0.08 \times 0.60} = 49{,}417 \text{ veh/day}$$

$$DSV_B = \frac{3{,}850}{0.08 \times 0.60} = 80{,}208 \text{ veh/day}$$

$$DSV_C = \frac{5{,}205}{0.08 \times 0.60} = 108{,}438 \text{ veh/day}$$

$$DSV_D = \frac{6{,}406}{0.08 \times 0.60} = 133{,}458 \text{ veh/day}$$

$$DSV_E = \frac{7{,}392}{0.08 \times 0.60} = 154{,}000 \text{ veh/day}$$

Discussion

These results can be conveniently shown in the form of a table, as illustrated in Exhibit 11-22. Given the approximate nature of these computations and the default values used, it is appropriate to round the DSV values to the nearest 100 veh/day, and SF and SV values to the nearest 10 veh/h.

LOS	*SF* (veh/h)	*SV* (veh/h)	*DSV* (veh/day)
A	2,730	2,370	49,400
B	4,430	3,850	80,200
C	5,980	5,210	108,400
D	7,360	6,410	133,500
E	8,500	7,390	154,000

Exhibit 11-22
Service Flow Rates, Service Volumes, and Daily Service Volumes for Example Problem 6

Exhibit 11-22, of course, applies only to the basic freeway segment as described. Should any of the prevailing conditions change, the values in the exhibit would also change. However, for a given segment, forecast demand volumes, whether given as flow rates, hourly volumes, or AADTs, could be compared with the criteria in Exhibit 11-22 to determine the likely LOS immediately. For example, if the 10-year forecast AADT for this segment is 125,000 veh/day, the expected LOS would be D.

5. REFERENCES

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

1. Schoen, J. A., A. May, W. Reilly, and T. Urbanik. *Speed-Flow Relationships for Basic Freeway Sections.* Final Report, NCHRP Project 3-45. JHK & Associates, Tucson, Ariz., May 1995.
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7. Hall, F. L., and L. M. Hall. Capacity and Speed-Flow Analysis of the Queen Elizabeth Way in Ontario. In *Transportation Research Record 1287,* Transportation Research Board, National Research Council, Washington, D.C., 1990, pp. 108–118.
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12. Webster, N., and L. Elefteriadou. A Simulation Study of Truck Passenger Car Equivalents (PCE) on Basic Freeway Segments. *Transportation Research,* Vol. 33B, No. 5, 1999, pp. 323–336.
13. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses.* Transportation Research Board of the National Academies, Washington, D.C., 2008.

APPENDIX A: COMPOSITE GRADES

In a basic freeway segment analysis, an overall average grade can be substituted for a series of grades if no single portion of the grade is steeper than 4% or the total length of the grade is less than 4,000 ft. For grades outside these limits (i.e., a portion of the grade is greater than 4% *and* the total length of the grade is greater than or equal to 4,000 ft), the composite grade procedure presented in this appendix is recommended. The composite grade procedure is used to determine an equivalent grade that will result in the same final speed of trucks as would the series of grades making up the composite.

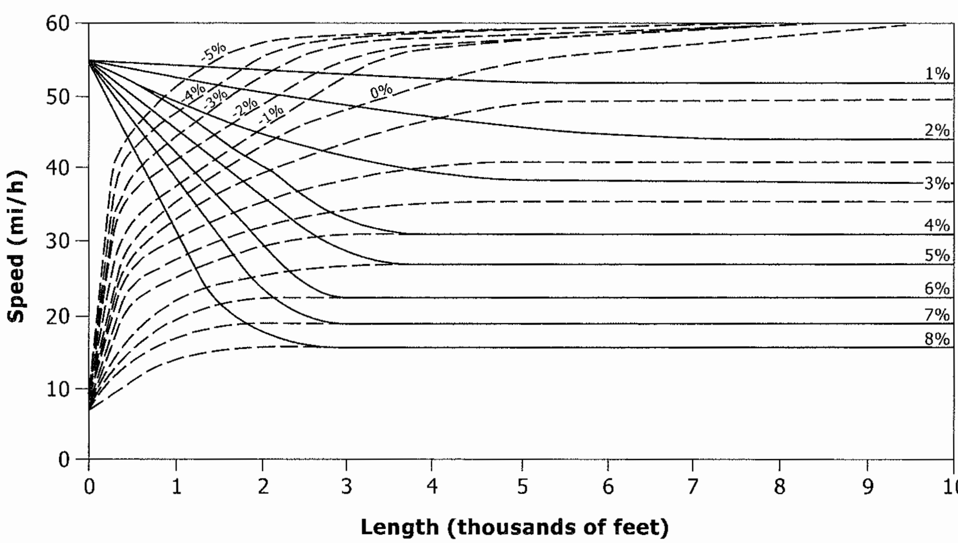
The composite grade procedure should be used for a series of grades that are ≥4,000 ft in length and that have a portion of the grade steeper than 4%.

The procedure finds the equivalent single grade that results in the same final truck speed as the series of grades would.

The acceleration and deceleration curves presented here are for vehicles with an average weight-to-horsepower ratio of 200 lb/hp, heavier than typical trucks found on freeways, which range between 125 lb/hp and 150 lb/hp. This is done in recognition of the fact that heavier trucks will have more of an impact on the traffic stream than lighter trucks.

Exhibit 11-A1 shows typical acceleration (*dashed lines*) and deceleration (*solid lines*) performance for a truck with a ratio of 200 lb/hp. The curves are conservative in that they assume a maximum truck speed of 55 mi/h for trucks entering a grade and 60 mi/h for trucks accelerating on a grade.

Exhibit 11-A1
Performance Curves for 200-lb/hp Truck



EXAMPLE PROBLEM

An example is provided to illustrate the process involved in determining an equivalent grade for a composite grade on a freeway. The example has two segments, but the procedure is valid for any number of segments. The composite grade is

- Upgrade of 2% for 5,000 ft, followed by
- Upgrade of 6% for 5,000 ft.

This grade should not be analyzed with an average grade approach, because one portion of the grade is steeper than 4% and the total length of the grade is in excess of 4,000 ft. As a comparison, application of the average grade approach in this case would yield the following:

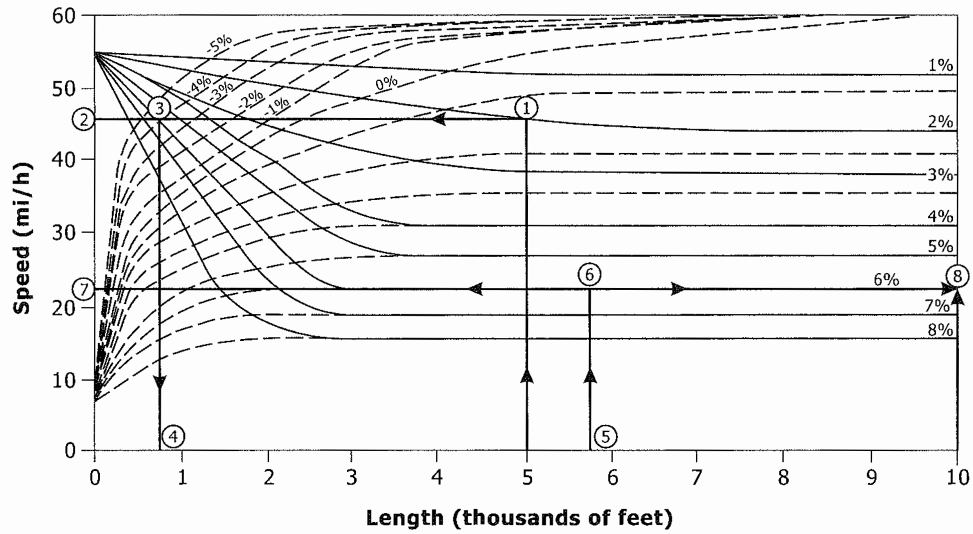
- Total rise along composite: (5,000 × 0.02) + (5,000 × 0.06) = 400 ft.
- Average grade: 400/10,000 = 0.04, or 4%.

With the average grade approach, the composite would be analyzed as if it were a single upgrade of 4%, 10,000 ft (1.89 mi) long.

Exhibit 11-A2 illustrates the recommended solution.

Exhibit 11-A2
Solution Using Composite Grade Procedure

The flat portions of the upgrade curves indicate the truck crawl speed for that grade.



A vertical line is drawn at 5,000 ft to the intersection with the curve for the +2% grade (Point 1). A horizontal line is drawn from the intersection point to the y-axis (Point 2). This procedure indicates that after 5,000 ft of +2% upgrade, trucks will be operating at a speed of approximately 46 mi/h.

This speed is also the speed at which trucks enter the +6% segment of the composite grade. The intersection of the 46-mi/h horizontal line with the curve for the +6% grade (Point 3) is found. A vertical line is dropped from this point to the x-axis (Point 4). This procedure indicates that trucks enter the +6% segment of the composite as if they had already been on the +6% grade for approximately 800 ft. Trucks will travel another 5,000 ft along the +6% grade, starting from Point 4. A vertical line is drawn at a distance of 800 + 5,000 = 5,800 ft (Point 5) to the intersection with the curve for the +6% grade (Point 6). A horizontal line drawn from this point to the y-axis (Point 7) indicates that the speed of trucks at the end of the two-segment composite grade will be approximately 23 mi/h.

The solution point is found as the intersection of a vertical line drawn at 10,000 ft (the total length of the composite grade) and a horizontal line drawn at 23 mi/h. The solution is read as the percent grade on which the solution point lies (Point 8). In this case, the point lies exactly on the curve for the 6% grade. Interpolations between curves are permissible.

In this case, the grade that is equivalent to the composite grade is a single grade of 6%, 10,000 ft (1.89 mi) long. This grade is 2% higher than the 4% average grade. The appropriate equivalent grade is the same percentage as the second segment of the composite grade because trucks have already reached crawl speed. Once trucks hit crawl speed, it does not matter how far from the beginning of the grade they are; their speed will remain constant.

PROCEDURAL STEPS

The general steps taken in solving for a composite-grade equivalent are summarized as follows:

1. Enter Exhibit 11-A1 with the length of the first segment of the composite grade.
2. Find the truck speed at the end of the first segment of the grade.
3. Find the length along the second segment of the grade that results in the same speed as that found in Step 2.
4. Add the length of the Segment 2 grade to the length determined in Step 3.
5. Repeat Steps 2 through 4 for each subsequent grade segment.
6. Find the intersection of a vertical line drawn at the total length of the composite grade and a horizontal line drawn at the final speed of trucks at the end of the composite grade.
7. Determine the percent of grade for the solution point of Step 6.

DISCUSSION

In the analysis of composite grades, the point of interest is not always at the end of the grade. It is important to identify the point at which the speed of trucks is the lowest because this is where trucks will have the maximum impact on operating conditions. This point may be an intermediate point. If a +3% grade of 1,000 ft is followed by a +4% grade of 2,000 ft, then by a +2% grade of 1,500 ft, the speed of trucks will be slowest at the end of the +4% grade segment. Thus, a composite grade solution would be sought for the first two segments of the grade, with a total grade length of 1,000 + 2,000 = 3,000 ft.

The composite grade procedure is not applicable in all cases, especially if the first segment is a downgrade and the segment length is long or if the segments are too short. In the use of performance curves, cases that cannot be solved with this procedure will become apparent to the analyst because the line will not intersect or the points will fall outside the limits of the curves. In such cases, field measurements of speeds should be used as inputs to the selection of appropriate truck equivalency values.

CHAPTER 12
FREEWAY WEAVING SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

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.

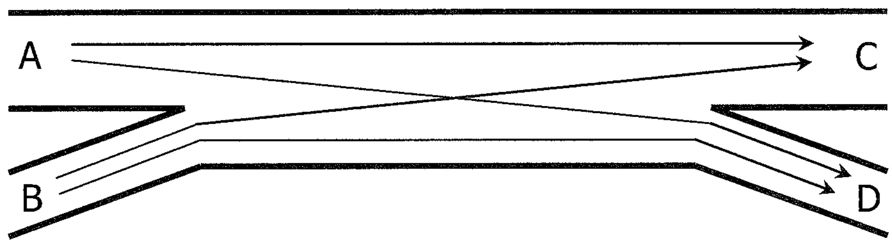
Three geometric characteristics affect a weaving segment's operating characteristics: length, width, and configuration. All have an impact on the critical lane-changing activity, which is the unique operating feature of a weaving segment. **Chapter 12, Freeway Weaving Segments,** provides a methodology for analyzing the operation of weaving segments based on 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 and provides examples of these applications.

2. WEAVING SEGMENT CHARACTERISTICS

OVERVIEW

Exhibit 12-1 illustrates a freeway weaving segment. 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. Flows A–D and B–C are, therefore, referred to as *weaving movements*. Flows A–C and B–D may also exist, but as they are not required to cross the path of any other flow, they are referred to as *nonweaving movements*.

Exhibit 12-1
Formation of a Weaving Segment



Traffic in a weaving segment experiences more lane-changing turbulence than is normally present on basic freeway segments.

Weaving segments require intense lane-changing maneuvers as 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. This additional turbulence presents operational problems and design requirements, which are addressed by this chapter's methodology.

A weaving segment's geometry affects its operating characteristics.

Three geometric characteristics affect a weaving segment's operating characteristics:

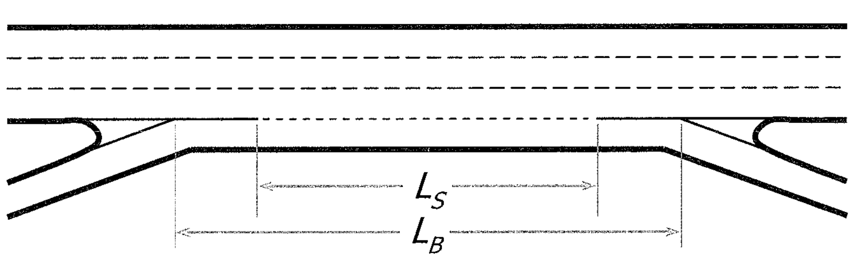
- Length,
- Width, and
- Configuration.

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 12-2.

Exhibit 12-2
Measuring the Length of a Weaving Segment



The lengths illustrated are defined as follows:

L_S = short length, the distance in feet between the end points of any barrier markings (solid white lines) that prohibit or discourage lane changing.

L_B = base length, the distance in feet between points in the respective gore areas where the left edge of the ramp-traveled way and the right edge of the freeway-traveled way meet.

Neither of these definitions is the same as those used in previous editions of the *Highway Capacity Manual* (HCM). The definitions used throughout the HCM2000 were historically tied to the specifics of the design of loop ramps in a cloverleaf interchange at a time when most weaving segments were part of such interchanges. Modern weaving segments occur in a wide range of situations and designs, and a more general definition of length is appropriate.

This methodology includes several equations that include the length of the weaving segment. In all cases, these equations use the short length L_S. This is not to suggest that lane changing in a weaving segment is restricted to this length. Some lane changing takes place over solid white lines and even painted gore areas. Nevertheless, research has shown that the short length is a better predictor of operating characteristics within the weaving segment than either the base length or the length as defined in HCM2000 and previous editions.

The weaving segment length used in the methodology is defined by the distance between barrier markings. Where no markings exist, the length is defined by the distance between where the left edge of the ramp-traveled way and the right edge of the freeway-traveled way meet.

For weaving segments in which no solid white lines are used, the two lengths illustrated in Exhibit 12-2 are the same, that is, $L_S = L_B$. In dealing with future designs in which the details of markings are unknown, a default value should be based on the general marking policy of the operating agency. At the time this methodology was developed, where solid white lines were provided, L_S was equal to $0.77 \times L_B$ on average for the available data.

The estimated speeds and densities, however, apply over the base length L_B. Some evidence also indicates that these speeds and densities may apply to the 500 ft of freeway upstream of the merge and downstream of the diverge because of presegregation of movements in each case.

The weaving segment length strongly influences lane-changing intensity. For any given demand situation, longer segments allow weaving motorists more time and space to execute their lane changes. This reduces the density of lane changing and, therefore, turbulence. Lengthening a weaving segment both increases its capacity and improves its operation (assuming a constant demand).

Under constant demand conditions, making a weaving segment longer increases its capacity and improves its operation.

WIDTH OF A WEAVING SEGMENT

The width of a weaving segment is measured as the number of continuous lanes within the segment, that is, the number of continuous lanes between the entry and exit gore areas. Acceleration or deceleration lanes that extend partially into the weaving segment are not included in this count.

The number of continuous lanes between gore areas within a weaving segment defines its width.

While additional lanes provide more space for both weaving and nonweaving vehicles, they encourage additional optional lane-changing activity. Thus, while reducing overall densities, additional lanes can increase lane-changing activity and intensity. In most cases, however, the number of lanes in the weaving segment is controlled by the number of lanes on the entry and exit legs and the intended configuration.

CONFIGURATION OF A WEAVING SEGMENT

Configuration of a weaving segment refers to the way that entry and exit lanes are linked. The configuration determines how many lane changes a weaving driver must make to complete the weaving maneuver successfully. The following sections use a great deal of terminology to describe configurations; this terminology should be clearly understood.

One-Sided and Two-Sided Weaving Segments

One-sided weaving segments require no more than two lane changes to complete a weaving maneuver.

Most weaving segments are one-sided. In general, this means that the ramps defining the entry to and exit from the weaving segment are on the same side of the freeway—either both on the right (most common) or both on the left. The methodology of this chapter was developed for one-sided weaving segments; however, guidelines are given for applying the methodology to two-sided weaving segments.

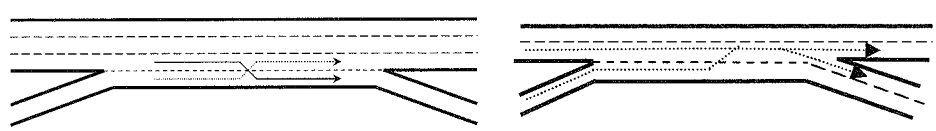
Two-sided weaving segments require three or more lane changes to complete a weaving maneuver or have a single-lane on-ramp closely followed by a single-lane off-ramp on the opposite side of the freeway.

One- and two-sided weaving segments are defined as follows:

- A *one-sided weaving segment* is one in which no weaving maneuvers require more than two lane changes to be completed successfully.
- A *two-sided weaving segment* is one in which at least one weaving maneuver requires three or more lane changes to be completed successfully or in which a single-lane on-ramp is closely followed by a single-lane off-ramp on the opposite side of the freeway.

Exhibit 12-3 illustrates two examples of one-sided weaving segments.

Exhibit 12-3
One-Sided Weaving Segments Illustrated

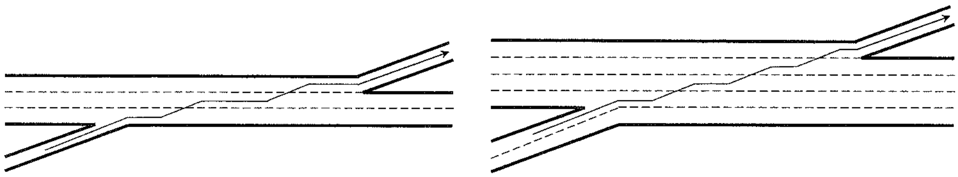


(a) One-Sided Ramp Weave (b) One-Sided Major Weave

Exhibit 12-3(a) shows a typical one-sided weaving segment formed by a one-lane, right-side on-ramp followed closely by a one-lane, right-side off-ramp. The two are connected by a continuous freeway auxiliary lane. Every weaving vehicle must make one lane change as illustrated, and the lane-changing turbulence caused is clearly focused on the right side of the freeway. Exhibit 12-3(b) shows another one-sided weaving segment in which the off-ramp has two lanes. One weaving movement (ramp to freeway) requires one lane change. The other (freeway to ramp) can be made without making a lane change. Again, lane-changing turbulence is focused on the right side of the freeway.

Exhibit 12-4 contains two examples of two-sided weaving segments.

Exhibit 12-4
Two-Sided Weaving Segments Illustrated



(a) Two-Sided Weaving Section with Single-Lane Ramps

(b) Two-Sided Weaving Section with Three Lane Changes

Exhibit 12-4(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 through movement on the freeway technically qualifies as a weaving flow.

Exhibit 12-4(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 Versus Major Weave Segments

Exhibit 12-3 can also be used to illustrate the difference between a ramp-weaving segment and a major weaving segment. Exhibit 12-3(a) shows a typical ramp-weaving segment, 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 13) and not as weaving segments.

It is important to note that 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 by using the methodology described in Chapter 13. The distance between the on-ramp and the off-ramp is not a factor in this determination.

Exhibit 12-3(b) shows a typical major weaving segment. A major weaving segment is formed when three or more entry or exit legs have multiple lanes.

Numerical Measures of Configuration

Three numerical descriptors of a weaving segment characterize its configuration:

"Minimum number of lane changes" assumes vehicles position themselves when entering and exiting to make the least number of lane changes possible.

LC_{RF} = minimum number of lane changes that a ramp-to-freeway weaving vehicle must make to complete the ramp-to-freeway movement successfully.

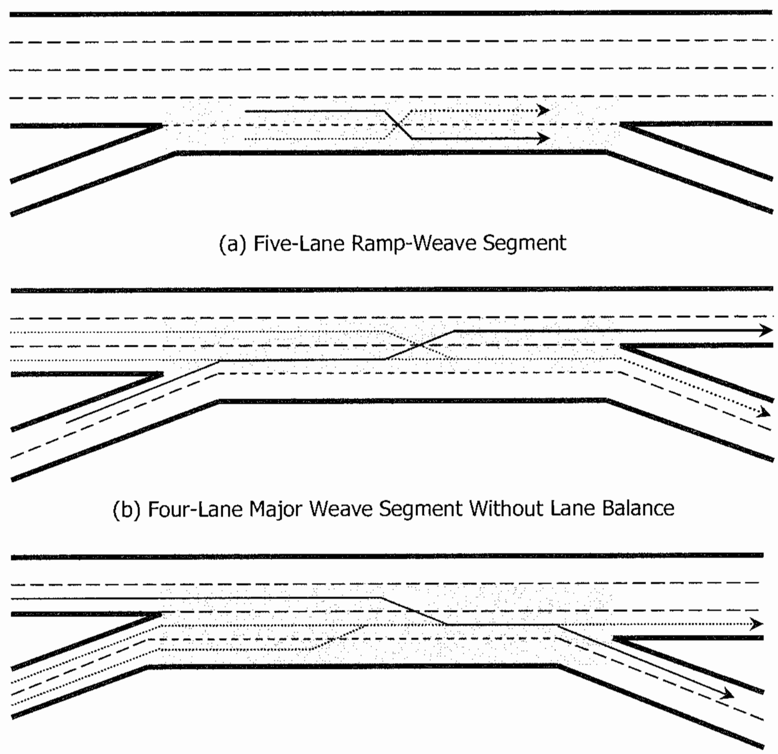
LC_{FR} = minimum number of lane changes that a freeway-to-ramp weaving vehicle must make to complete the freeway-to-ramp movement successfully.

N_{WL} = number of lanes from which a weaving maneuver may be completed with one lane change or no lane changes.

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. Exhibit 12-5 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 12-5
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 12-5(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 12-5(b) and Exhibit 12-5(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 12-5(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 12-5(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 12-5(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 12-5(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 $LC_{FR} = 1$.

In Exhibit 12-5(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 12-5(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 fewer 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.

Lane balance creates more flexibility in Exhibit 12-5(c). 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, $N_{WL} = 3$.

Only the ramp-to-ramp movement is considered to be a weaving flow in a two-sided weaving segment.

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.

Special Case: Two-Sided Weaving Segments

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 12-4(a), $LC_{RR} = 2$, while in Exhibit 12-4(b), $LC_{RR} = 3$.
2. In all cases of two-sided weaving, the value of N_{WL} is set to 0 by definition.

With these two modifications, the methodology outlined for one-sided weaving segments may be applied to two-sided weaving segments as well.

3. METHODOLOGY

The methodology presented in this chapter was developed as part of National Cooperative Highway Research Program (NCHRP) Project 3-75, *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*).

LIMITATIONS OF THE METHODOLOGY

The methodology of this chapter does not specifically address the following subjects (without modifications by the analyst):

- Special lanes, such as high-occupancy vehicle lanes, within the weaving segment;
- Ramp metering on entrance ramps forming part of the weaving segment;
- Specific operating conditions when oversaturated conditions exist;
- Effects of speed limit enforcement practices on weaving segment operations;
- Effects of intelligent transportation system technologies on weaving segment operations;
- Weaving segments on arterials or other urban streets, including one-way frontage roads;
- Effects of downstream congestion or upstream demand starvation on the analysis segment; or
- Multiple weaving segments.

The last subject has been included in previous versions of this manual. Multiple weaving segments must now be divided into appropriate merge, diverge, and simple weaving segments for analysis.

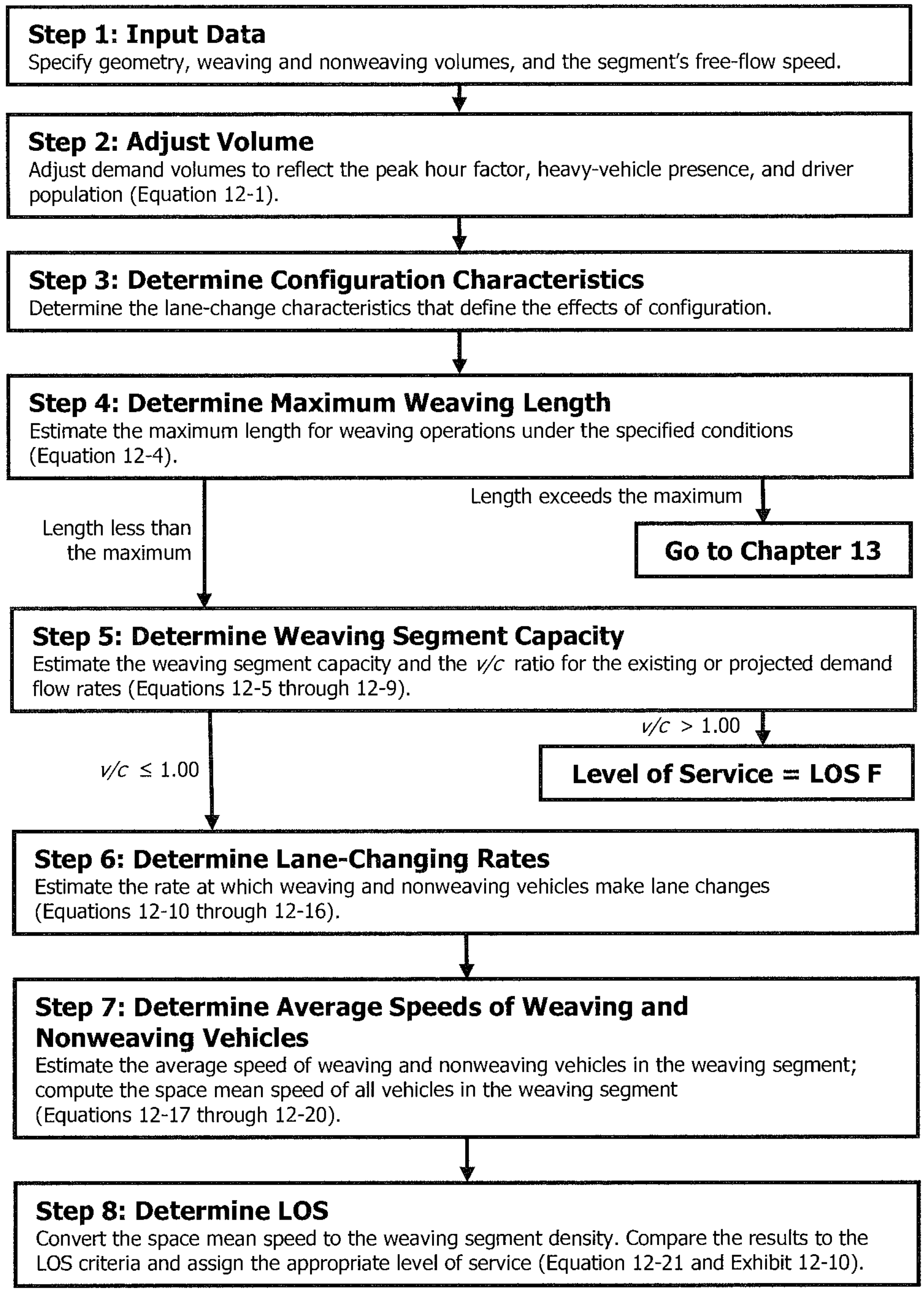
Multiple weaving segments must be divided into merge, diverge, and simple weaving segments for analysis.

OVERVIEW OF THE METHODOLOGY

Exhibit 12-6 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 Level of Service (LOS) F.
- Models to predict the capacity of a weaving segment under both ideal and prevailing conditions.
- A model to estimate the maximum length over which weaving operations can be said to exist.

Exhibit 12-6
Weaving Methodology Flowchart



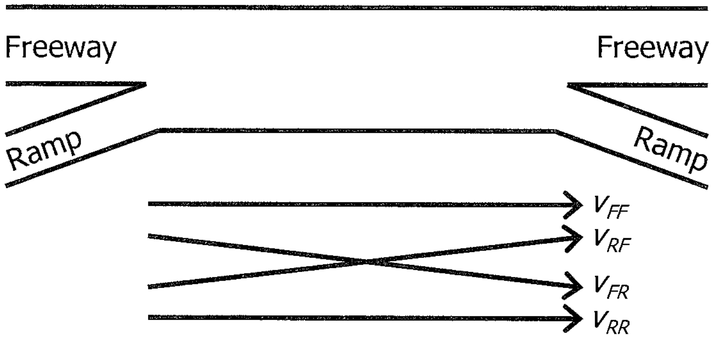
If the potential weaving segment is longer than the value given by Equation 12-4, it is treated as isolated merging and diverging ramp junctions by using the procedures of Chapter 13.

LOS F exists in a weaving segment when demand exceeds capacity.

PARAMETERS DESCRIBING A WEAVING SEGMENT

Several parameters describing weaving segments have already been introduced and defined. Exhibit 12-7 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 these apply only to one-sided weaving segments. Exhibit 12-8 lists those variables that are different when applied to two-sided weaving segments.

Exhibit 12-7
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, v_W/v;

N = number of lanes within the weaving section;

N_{WL} = number of lanes from which a weaving maneuver may be made with one or no lane changes (see Exhibit 12-5);

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 12-2;

LC_{RF} = minimum number of lane changes that must be made by a single weaving vehicle moving from the on-ramp to the freeway (see Exhibit 12-5);

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_{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);

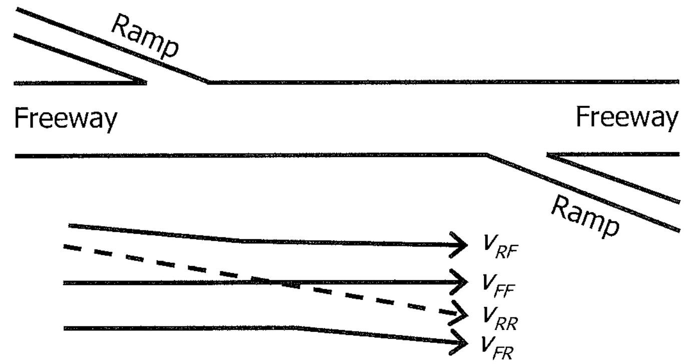
LC_{NW} = total rate of lane changing by nonweaving vehicles within the weaving segment (lc/h);

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 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 12-8
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 12-7, 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 PROCEDURES

Each of the major procedural steps noted in Exhibit 12-6 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 LOS that is expected to exist.

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.

Traffic demands are usually expressed as peak hour volumes under prevailing conditions. If flow rates have been directly observed in the field, the flow rates for the worst 15-min period in the peak hour may be substituted. In this case, the peak hour factor (PHF) is implicitly 1.00.

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 12-1:

$$v_i = \frac{V_i}{PHF \times f_{HV} \times f_p}$$

Equation 12-1

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;

f_{HV} = adjustment factor for heavy-vehicle presence; and

f_p = adjustment factor for driver population; 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.

Factors f_{HV} and f_p are taken from Chapter 11, Basic Freeway 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. If hourly volumes are converted by using a PHF other than 1.00, there is an implicit assumption that all four component flows in the weaving segment peak during the same 15-min period of the hour. This is rarely true in the field; however, such an analysis represents a worst-case scenario.

Once demand flow rates have been established, it may be convenient to construct a weaving diagram similar to those illustrated in Exhibit 12-7 (for one-sided weaving segments) and Exhibit 12-8 (for two-sided weaving segments).

Step 3: Determine Configuration Characteristics

Several key parameters characterize the configuration of a weaving segment. These are descriptive of the segment and will be used as key 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.

How these values are determined depends on whether the segment under study is a one-sided or two-sided weaving segment.

One-Sided Weaving Segments

The determination of key variables in one-sided weaving segments is illustrated in Exhibit 12-7. In one-sided segments, the two weaving movements are the ramp-to-freeway and freeway-to-ramp flows. As shown in Exhibit 12-7, 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, 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_{MIN} for one-sided weaving segments is given by Equation 12-2:

Equation 12-2

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

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 12-5.

Two-Sided Weaving Segments

The determination of key variables in two-sided weaving segments is illustrated in Exhibit 12-8. The unique feature of two-sided weaving segments is that only the ramp-to-ramp flow is functionally weaving. From Exhibit 12-8, 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_{MIN} for two-sided weaving segments is given by Equation 12-3:

$$LC_{MIN} = LC_{RR} \times v_{RR}$$

Equation 12-3

For two-sided weaving segments, the value of N_{WL} is always 0 by definition.

Step 4: Determine Maximum Weaving Length

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.

The maximum length of a weaving segment, L_{MAX}, is based on the distance beyond which additional length does not add to capacity.

Unfortunately, depending on the selected definition, these measures can be quite different. Weaving turbulence will have an impact on operations (i.e., weaving and nonweaving vehicle speeds) for distances far in excess of those defined by when the capacity of the segment 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 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

Equation 12-4

where L_{MAX} is the maximum weaving segment length (using the short length definition) and other variables are as previously defined.

As VR increases, it is expected that the influence of weaving turbulence would 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 12-9 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. While values of VR above 0.70 are technically feasible on segments with N_{WL} = 3.0 lanes, they are rare.

While the extreme values in Exhibit 12-9 are not practical, it is clear that the maximum length of weaving segments can rise to 6,000 ft or more. Furthermore, the maximum length can vary over time, as VR is not a constant throughout every demand period of the day.

Exhibit 12-9
Variation of Weaving Length Versus Volume Ratio and Number of Weaving Lanes (ft)

	Number of Weaving Lanes	
VR	$N_{WL} = 2$	$N_{WL} = 3$
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

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 13. Any portion falling outside the influence of the merge and diverge segments is treated as a basic freeway segment.

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 13.

If the segment is too long to be considered a weaving segment, then the merge and diverge areas are treated separately. Any distance between the two falling outside the influence areas of the merge and diverge segments would be considered to be a basic freeway segment and would be 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:

- 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
- 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 criterion is based on the criteria listed in Chapter 11, Basic Freeway Segments, which state that freeway 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 criterion recognizes that there is a practical limit to how many vehicles can actually 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.

For two-sided weaving segments ($N_{WL} = 0$ lanes), no limiting value on weaving flow rate is proposed. The analysis of two-sided weaving segments is approximate with this methodology, and a density sufficient to cause a breakdown is generally reached at relatively low weaving flow rates.

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 12-5:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_S\right] + \left[119.8N_{WL}\right]$$

Equation 12-5

where

c_{IWL} = capacity of the weaving segment under equivalent ideal conditions, per lane (pc/h/ln), and

c_{IFL} = capacity of a basic freeway segment with the same FFS as the weaving segment under equivalent ideal conditions, per lane (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, as both increasing length and a larger number of weaving lanes reduce the intensity of turbulence.

Arithmetically, it is possible to get a result in which c_{IWL} is greater than c_{IFL}. In practical terms, 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 12-6:

$$c_W = c_{IWL} N f_{HV} f_p$$

Equation 12-6

where c_W is the capacity of the weaving segment under prevailing conditions in vehicles per hour. As with all capacities, it is stated as a flow rate for a 15-min analysis period.

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 12-7:

$$c_{IW} = \frac{2{,}400}{VR} \quad \text{for } N_{WL} = 2 \text{ lanes}$$

$$c_{IW} = \frac{3{,}500}{VR} \quad \text{for } N_{WL} = 3 \text{ lanes}$$

Equation 12-7

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 must be converted to prevailing conditions by using Equation 12-8:

$$c_W = c_{IW} f_{HV} f_p$$

Equation 12-8

Final Determination of Capacity

The final capacity is the smaller of the two estimates of Equation 12-6 and Equation 12-8. With capacity determined, a v/c ratio for the weaving segment may be computed from Equation 12-9:

Equation 12-9

$$v/c = \frac{v f_{HV} f_p}{c_W}$$

Adjustment factors are used because the total demand flow rate, v, is stated for equivalent ideal conditions, while c_W is stated for prevailing conditions.

LOS F occurs when demand exceeds capacity.

Level of Service F

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, it is expected that queues will 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. Where LOS F is found to exist, the analyst is urged to use the methodology of Chapter 10, Freeway Facilities, to analyze the impacts of this on upstream and downstream segments during the analysis period and over time.

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 determine the existing or anticipated LOS.

It should be noted that the lane-changing rates estimated are in terms of equivalent *passenger-car lane changes*. It is assumed that heavy-vehicle lane changes 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 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 12-10:

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5} N^2 (1 + ID)^{0.8}\right]$$

Equation 12-10

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; and

ID = interchange density (int/mi).

Equation 12-10 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. In using Equation 12-10, however, 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 is the first use of interchange density in a model not involving determination of the FFS. In this edition of the HCM, however, FFS 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 when determining the value of *total ramp density* and *interchange density*, as they are different numbers.

The algorithm uses the term $1 + ID$ because the value of ID may be either 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. The subject weaving segment should be counted as one interchange in this computation. For additional discussion of total ramp density, consult Chapter 11.

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 upon 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. These are more difficult to predict than weaving lane changes, as 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 (*10*) 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 12-11:

Equation 12-11

$$I_{NW} = \frac{L_S \times ID \times v_{NW}}{10{,}000}$$

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 seem 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 12-12, 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:

Equation 12-12

$$LC_{NW1} = (0.206 v_{NW}) + (0.542 L_s) - (192.6N)$$

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 12-12 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 produce extraordinarily high nonweaving lane-changing rates. Equation 12-13 is used in cases for which I_{NW} is greater than or equal to 1,950:

Equation 12-13

$$LC_{NW2} = 2{,}135 + 0.223(v_{NW} - 2{,}000)$$

where LC_{NW2} is the lane-changing rate per hour, and all other variables are as previously defined.

Unfortunately, Equation 12-12 and Equation 12-13 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 12-14:

$$LC_{NW3} = LC_{NW1} + \left(LC_{NW2} - LC_{NW1}\right)\left(\frac{I_{NW} - 1{,}300}{650}\right)$$

Equation 12-14

where LC_{NW3} is the lane-changing rate per hour, and all other variables are as previously defined. Equation 12-14 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 12-15 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 12-15

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 12-16:

$$LC_{ALL} = LC_W + LC_{NW}$$

Equation 12-16

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 12-17:

$$S_W = S_{MIN} + \left(\frac{S_{MAX} - S_{MIN}}{1 + W}\right)$$

Equation 12-17

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.

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. As with all analyses, the FFS is best observed in the field, either on the subject facility or a similar facility. When measured, the FFS should be observed within the weaving segment.

In situations that require the FFS to be estimated, the model described in Chapter 11, Basic Freeway Segments, is used. The average speed of weaving vehicles within the weaving segment is estimated by using Equation 12-18 and Equation 12-19:

Equation 12-18

$$S_W = 15 + \left(\frac{FFS - 15}{1 + W} \right)$$

Equation 12-19

$$W = 0.226 \left(\frac{LC_{ALL}}{L_S} \right)^{0.789}$$

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 12-20:

Equation 12-20

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048 \frac{v}{N} \right)$$

Equation 12-20 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 12-21:

Equation 12-21

$$S = \frac{v_W + v_{NW}}{\left(\frac{v_W}{S_W} \right) + \left(\frac{v_{NW}}{S_{NW}} \right)}$$

Step 8: Determine LOS

The LOS in a weaving segment, as in all freeway analysis, is related to the density in the segment. Exhibit 12-10 provides LOS criteria for weaving segments on freeways, collector–distributor (C-D) roadways, 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.

LOS can be determined for weaving segments on freeways, multilane highways, and C-D roadways.

Exhibit 12-10
LOS for Weaving Segments

LOS	Density (pc/mi/ln)	
	Freeway Weaving Segments	**Weaving Segments on Multilane Highways or C-D Roadways**
A	0–10	0–12
B	>10–20	>12–24
C	>20–28	>24–32
D	>28–35	>32–36
E	>35	>36
F	Demand exceeds capacity	

The boundary between stable and unstable flow—the boundary between levels of service E and F—occurs when the demand flow rate exceeds the capacity of the segment, as described in Step 5. 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). It is believed that drivers will tolerate higher densities in an area where lane-changing turbulence is expected than on basic segments.

To apply density criteria, the average speed of all vehicles, computed in Step 7, must be converted to density by using Equation 12-22.

$$D = \frac{\left(\frac{v}{N}\right)}{S}$$

Equation 12-22

where D is density in passenger cars per mile per lane and all other variables are as previously defined.

SPECIAL CASES

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 analyzed as separate merge, diverge, and simple weaving segments, as appropriate.

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 13, Freeway Merge and Diverge Segments. Chapter 11, Basic Freeway Segments, contains information relative to the process of identifying appropriate segments for analysis.

C-D Roadways

The methodology applies approximately to C-D roadways, 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 roadways 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 roadway. It would have to be measured on an existing or similar C-D roadway, as the predictive methodology of FFS given in Chapter 11 does not apply to such roadways. It is less clear that the LOS criteria of Exhibit 12-10 are appropriate. Many C-D roadways operate at lower speeds and higher densities than on basic segments, and the criteria of Exhibit 12-10 may produce an inappropriately negative view of operations on a C-D roadway.

If the measured FFS of a C-D roadway is high (greater than or equal to 50 mi/h), the results of analysis can be expected to be reasonably accurate. 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 surface 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.

Arterial Weaving

No generally accepted analysis methodologies currently exist for arterial weaving movements.

The methodology of this chapter does not apply to weaving segments on arterials. Arterial weaving is strongly affected by the proximity and timing of signals along the arterial. At the present time, there are no generally accepted analytic methodologies for analyzing weaving movements on arterials.

4. APPLICATIONS

The methodology of this chapter 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. Other types of analysis, however, are possible.

DEFAULT VALUES

An NCHRP report (*10*) provides a comprehensive presentation of potential default values for uninterrupted-flow facilities. Default values for freeways are summarized in Chapter 10, Freeway Facilities. These defaults cover the key characteristics of PHF and percentage of heavy vehicles. Recommendations are based on geographical region, population, and time of day. All general freeway default values may be applied to the analysis of weaving segments in the absence of field data or projected conditions.

There are many specific variables related to weaving segments. It is, therefore, virtually impossible to specify default values of such characteristics as length, width, configuration, and balance of weaving and nonweaving flows. Weaving segments are a detail of the freeway design and should therefore be treated only with the specific characteristics of the segment known or projected. Small changes in some of these variables can and do yield significant changes in the analysis results.

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 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 generally 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 the planning and preliminary design phase, however, demand flows are generally stated as average annual daily traffic (AADT) statistics that must be converted to directional design hour volumes. A number of variables may be unknown (e.g., PHF and percentage of heavy vehicles); these may be replaced by default values.

Service Flow Rates, Service Volumes, and Daily Service Volumes

This manual defines three sets of values that are related to LOS boundary conditions:

SF_i = service flow rate for LOS i (veh/h),

SV_i = service volume for LOS i (veh/h), and

DSV_i = daily service volume for LOS i (veh/day).

The *service flow rate* is the maximum rate of flow (for a 15-min interval) that can be accommodated on a segment while maintaining all operational criteria for LOS i under prevailing roadway and traffic conditions. The *service volume* is the maximum hourly volume that can be accommodated on a segment while maintaining all operational criteria for LOS i during the worst 15 min of the hour under prevailing roadway and traffic conditions. The *daily service volume* is the maximum AADT that can be accommodated on a segment while maintaining all operational criteria for LOS i during the worst 15 min of the peak hour under prevailing roadway and traffic conditions. The service flow rate and service volume are unidirectional values, while the daily service volume is a total two-way volume. In the context of a weaving section, the daily service volume is highly approximate, as it is rare that both directions of a freeway have a weaving segment with similar geometry.

In general, service flow rates are initially computed for ideal conditions and are then converted to prevailing conditions by using Equation 12-23 and the appropriate adjustment factors from Chapter 11, Basic Freeway Segments:

Equation 12-23

$$SF_i = SFI_i \times f_{HV} \times f_p$$

where

SFI_i = service flow rate under ideal conditions (pc/h),

f_{HV} = adjustment factor for heavy-vehicle presence (Chapter 11), and

f_p = adjustment factor for driver population (Chapter 11).

The methodology of this chapter is used to determine the values of ideal service flow rate (SFI) for the specific weaving segment under study. The capacity of the segment is equivalent to the ideal service flow rate for LOS E. For other levels of service, the total flow rates required to produce threshold densities (Exhibit 12-10) are found. This is an iterative procedure in which all other characteristics are held constant. Iterative analyses are conducted until the defining densities are produced.

Once the ideal service flow rates are determined, service flow rates under prevailing conditions are computed by using Equation 12-23. These can be converted to hourly service volumes SV by using Equation 12-24. Service volumes can then be converted to daily service volumes DSV by using Equation 12-25.

$$SV_i = SF_i \times PHF$$

Equation 12-24

$$DSV_i = \frac{SV_i}{K \times D}$$

Equation 12-25

where

K = proportion of AADT occurring during the peak hour, and

D = proportion of traffic in the peak direction.

All other variables are as previously defined.

Example Problem 5 illustrates the computation of service flow rates, service volumes, and daily service volumes for a specific weaving segment.

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.

Strengths of the HCM Procedure

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 a body of 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 simulators 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 algorithms;

- Producing a single deterministic estimate of LOS, which is important for some purposes, such as development impact reviews; and
- Generating reproducible results with a small commitment of resources (including calibration) from a precisely documented methodology.

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

Weaving segments can be analyzed by using a variety of stochastic and deterministic simulation tools that address freeways. These tools can be very 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.

The limitations stated earlier in this chapter may be addressed by using available simulation tools. The following conditions, which are beyond the scope of this chapter, are treated explicitly by simulation tools:

- *Managed lanes within the weaving segment.* These lanes are typically modeled explicitly by simulation; for example, when one or more weaving movements are regulated by using pavement markings, signage, physical longitudinal barriers, or some combination of these.
- *Ramp metering on entrance ramps forming part of the weaving segment.* These features are also 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.
- *Effects of intelligent transportation system technologies on weaving segment operations.* Some intelligent transportation system features such as dynamic message signs are offered by a few simulation tools. Some features are modeled explicitly by simulation; others may be approximated by using assumptions (e.g., by modifying origin–destination demands by time interval).
- *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 Chapter 10 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

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.

In addition to offering more performance measures, alternative tools can identify specific point problems that could be overlooked in a segment-level analysis.

Development of HCM-Compatible Performance Measures Using Alternative Tools

When using alternative tools, 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

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.

Direct comparison of the numerical outputs from the HCM and alternative tools can be misleading.

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 to 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 later in this 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?

5. EXAMPLE PROBLEMS

Exhibit 12-11
List of Example Problems

Example Problem	Description	Application
1	LOS of a major weaving segment	Operational Analysis
2	LOS of a ramp-weaving segment	Operational Analysis
3	LOS of a two-sided weaving segment	Operational Analysis
4	Design of a major weaving segment for a desired LOS	Design
5	Service volume table construction	Service Volumes

EXAMPLE PROBLEM 1: LOS OF A MAJOR WEAVING SEGMENT

The Weaving Segment

The subject of this operational analysis is a major weaving segment on an urban freeway, as shown in Exhibit 12-12.

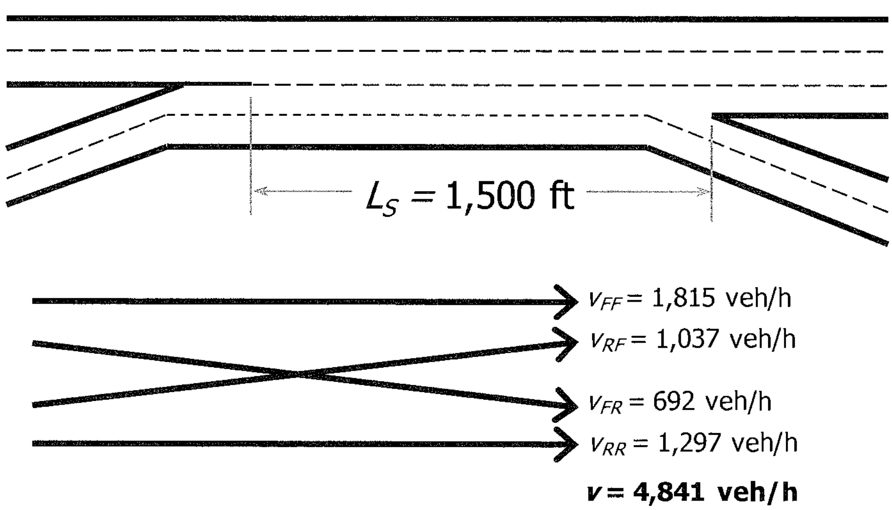


Exhibit 12-12
Major Weaving Segment for Example Problem 1

What is the LOS and capacity of the weaving segment shown in Exhibit 12-12?

The Facts

In addition to the information contained in Exhibit 12-12, the following characteristics of the weaving segment are known:

PHF = 0.91 (for all movements);

Heavy vehicles = 10% trucks, 0% recreational vehicles (RVs) (all movements);

Driver population = regular commuters;

FFS = 65 mi/h;

c_{IFL} = 2,350 pc/h/ln (for FFS = 65 mi/h);

ID = 0.8 int/mi; and

Terrain = level.

Comments

Chapter 11, Basic Freeway Segments, must be consulted to find appropriate values for the heavy-vehicle adjustment factor f_{HV} and the driver population adjustment factor f_p.

All input parameters have been specified, so default values are not needed. Demand volumes are given in vehicles per hour under prevailing conditions. These must be converted to passenger cars per hour under equivalent ideal conditions for use in equations of the methodology. The length of the segment must be compared with the maximum length for weaving analysis to determine whether the methodology of this chapter is applicable. The capacity of the weaving segment is estimated and compared with the total demand flow to determine whether LOS F exists. Lane-changing rates are estimated to allow speed estimates to be made for weaving and nonweaving flows. An average overall speed and density are computed and compared with the criteria of Exhibit 12-10 to determine LOS.

Step 1: Input Data

All inputs have been specified in Exhibit 12-12 and the Facts section of the problem statement.

Step 2: Adjust Volume

Equation 12-1 is used to convert the four component demand volumes to flow rates under equivalent ideal conditions. Chapter 11 is consulted to obtain a value of E_T (1.5 for level terrain) and f_p (1.00 for regular commuters). The heavy-vehicle adjustment factor is computed as

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_{RV}(E_{RV} - 1)} = \frac{1}{1 + 0.10(1.5 - 1)} = 0.952$$

Equation 12-1 is now used to convert all demand volumes:

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

$$v_{FF} = \frac{1{,}815}{0.91 \times 0.952 \times 1} = 2{,}094 \text{ pc/h}$$

$$v_{FR} = \frac{692}{0.91 \times 0.952 \times 1} = 798 \text{ pc/h}$$

$$v_{RF} = \frac{1{,}037}{0.91 \times 0.952 \times 1} = 1{,}197 \text{ pc/h}$$

$$v_{RR} = \frac{1{,}297}{0.91 \times 0.952 \times 1} = 1{,}497 \text{ pc/h}$$

Then

$$v_W = 798 + 1{,}197 = 1{,}995 \text{ pc/h}$$

$$v_{NW} = 2{,}094 + 1{,}497 = 3{,}591 \text{ pc/h}$$

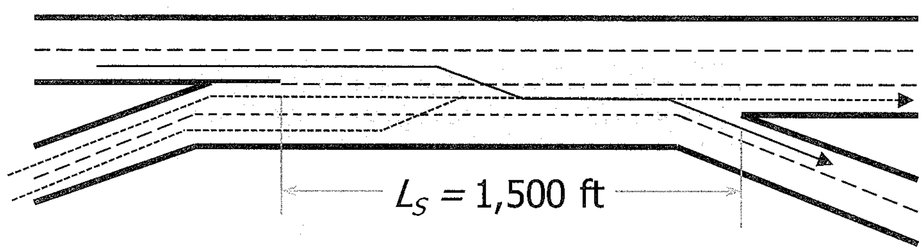
$$v = 1{,}995 + 3{,}591 = 5{,}586 \text{ pc/h}$$

$$VR = \frac{1{,}995}{5{,}586} = 0.357$$

Step 3: Determine Configuration Characteristics

The configuration is examined to determine the values of LC_{RF}, LC_{FR}, and N_{WL}. These determinations are illustrated in Exhibit 12-13. From these values, the minimum number of lane changes by weaving vehicles, LC_{MIN}, is then computed by using Equation 12-2.

Exhibit 12-13
Determination of Configuration Variables for Example Problem 1



From Exhibit 12-13, it can be seen that ramp-to-freeway vehicles can execute their weaving maneuver without making a lane change (if they so desire). Thus, $LC_{RF} = 0$. Freeway-to-ramp vehicles must make at least one lane change to complete their desired maneuver. Thus, $LC_{FR} = 1$. If optional lane changes are considered, weaving movements can be accomplished with one or no lane changes from both entering ramp lanes and from the rightmost freeway lane. Thus, $N_{WL} = 3$. Equation 12-2 can now be employed:

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR}) = (0 \times 1197) + (1 \times 798) = 798 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length

The maximum length over which weaving movements may exist is determined by Equation 12-4. The determination is case specific, and the result is valid only for the case under consideration:

$$L_{MAX} = \left[5728(1+VR)^{1.6}\right] - \left[1566 N_{WL}\right]$$

$$L_{MAX} = \left[5728(1+0.357)^{1.6}\right] - \left[1566 \times 3\right] = 4{,}639 \text{ ft}$$

As the maximum length is significantly greater than the actual segment length of 1,500 ft, weaving operations do exist, and the analysis may continue with the weaving analysis methodology.

Step 5: Determine Weaving Segment Capacity

Capacity may be controlled by one of two factors: operations reaching a maximum density of 43 pc/mi/ln or by the weaving demand flow rate reaching

3,500 pc/h (for a weaving segment with $N_{WL} = 3$). Equation 12-5 through Equation 12-8 are used to make these determinations.

Capacity Controlled by Density

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_s\right] + \left[119.8N_{WL}\right]$$

$$c_{IWL} = 2{,}350 - \left[438.2(1+0.357)^{1.6}\right] + \left[0.0765 \times 1{,}500\right] + \left[119.8 \times 3\right]$$

$$c_{IWL} = 2{,}110 \text{ pc/h/ln}$$

$$c_W = c_{IWL} N f_{HV} f_p = 2{,}110 \times 4 \times 0.952 \times 1 = 8{,}038 \text{ veh/h}$$

Capacity Controlled by Maximum Weaving Flow Rate

$$c_{IW} = \frac{3{,}500}{VR} = \frac{3{,}500}{0.357} = 9{,}800 \text{ pc/h}$$

$$c_W = c_{IW} f_{HV} f_p = 9{,}800 \times 0.952 \times 1 = 9{,}333 \text{ veh/h}$$

Note that the methodology computes the capacity controlled by density in passenger cars per hour per lane, while the capacity controlled by maximum weaving flow rate is computed in passenger cars per hour. After conversion, however, both are in units of vehicles per hour.

The controlling value is the smaller of these, or 8,038 veh/h. As the total demand flow rate is only 5,320 veh/h, the capacity is clearly sufficient, and this situation will not result in LOS F.

Step 6: Determine Lane-Changing Rates

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}(N^2)(1+ID)^{0.8}\right]$$

$$LC_W = 798 + 0.39\left[(1{,}500 - 300)^{0.5} 4^2 (1+0.8)^{0.8}\right] = 1{,}144 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{1{,}500 \times 0.8 \times 3{,}591}{10{,}000} = 431 < 1{,}300$$

$$LC_{NW} = (0.206 v_{NW}) + (0.542 L_S) - (192.6N)$$

$$LC_{NW} = (0.206 \times 3{,}591) + (0.542 \times 1{,}500) - (192.6 \times 4) = 782 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}144 + 782 = 1{,}926 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{1{,}926}{1{,}500}\right)^{0.789} = 0.275$$

Then

$$S_W = 15 + \left(\frac{FFS - 15}{1 + W}\right) = 15 + \left(\frac{65 - 15}{1 + 0.275}\right) = 54.2 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048\frac{v}{N}\right)$$

$$S_{NW} = 65 - (0.0072 \times 798) - \left(0.0048 \times \frac{5{,}586}{4}\right) = 52.5 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{3{,}591 + 1{,}995}{\left(\frac{3{,}591}{52.5}\right) + \left(\frac{1{,}995}{54.2}\right)} = 53.1 \text{ mi/h}$$

Step 8: Determine LOS

Equation 12-22 is used to convert the average speed of all vehicles in the segment to an average density:

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{5{,}586}{4}\right)}{53.1} = 26.3 \text{ pc/mi/ln}$$

The resulting density of 26.3 pc/mi/ln is compared with the LOS criteria of Exhibit 12-10. The LOS is C, as the density is within the specified range of 20 to 28 pc/h/ln for that level.

Discussion

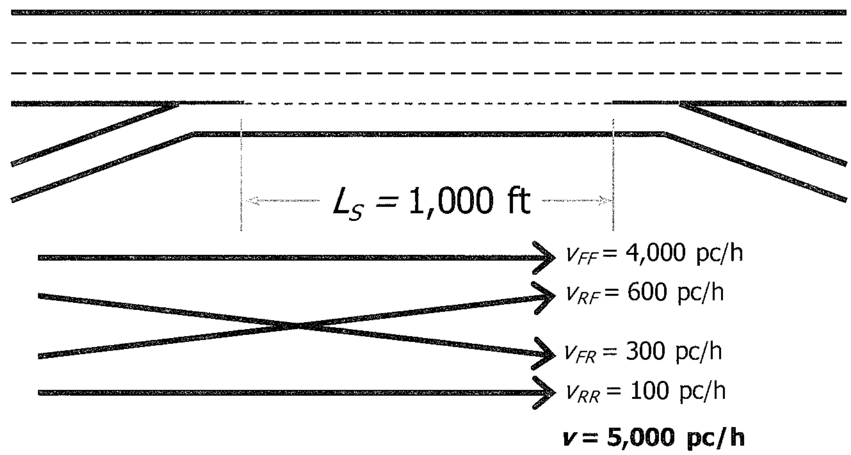
As indicated by the results, this weaving segment operates at LOS C, with an average speed of 53.1 mi/h for all vehicles. Weaving vehicles travel a bit faster than nonweaving vehicles, primarily because the configuration favors weaving vehicles, allowing many weaving maneuvers to be made without making a lane change. The demand flow rate of 4,841 veh/h is considerably less than the capacity of the segment, 8,038 veh/h. In other words, demand can grow significantly before reaching the capacity of the segment.

EXAMPLE PROBLEM 2: LOS OF A RAMP-WEAVING SEGMENT

The Weaving Segment

The weaving segment that is the subject of this operational analysis is shown in Exhibit 12-14. It is a typical ramp-weave segment.

Exhibit 12-14
Ramp-Weave Segment for Example Problem 2



What is the capacity of the weaving segment of Exhibit 12-14, and at what LOS is it expected to operate with the demand flow rates as shown?

The Facts

In addition to the information given in Exhibit 12-14, the following facts are known about the subject weaving segment:

PHF = 1.00 (demands stated as flow rates);

Heavy vehicles = 0% trucks, 0% RVs (demands given as passenger car equivalents);

Driver population = regular commuters;

FFS = 75 mi/h;

c_{IFL} = 2,400 pc/h/ln (for FFS = 75 mi/h);

ID = 1.0 int/mi; and

Terrain = level.

Comments

Because the demands have been specified as flow rates in passenger cars per hour under equivalent ideal conditions, Chapter 11 does not have to be consulted to obtain appropriate adjustment factors.

Several of the computational steps related to converting demand volumes to flow rates under equivalent ideal conditions are trivial, as demands are already specified in that form. Lane-changing characteristics will be estimated. The maximum length for weaving operations in this case will be estimated and compared with the actual length of the segment. The capacity of the segment will be estimated and compared with the demand to determine whether LOS F exists. If it does not, component flow speeds will be estimated and averaged. A density will be estimated and compared with the criteria of Exhibit 12-10 to determine the expected LOS.

Step 1: Input Data

All input data are stated in Exhibit 12-14 and the Facts section.

Step 2: Adjust Volume

Because all demands are stated as flow rates in passenger cars per hour under equivalent ideal conditions, no further conversions are necessary. Key volume parameters are as follows:

$$v_{FF} = 4{,}000 \text{ pc/h}$$

$$v_{FR} = 600 \text{ pc/h}$$

$$v_{RF} = 300 \text{ pc/h}$$

$$v_{RR} = 100 \text{ pc/h}$$

$$v_W = 600 + 300 = 900 \text{ pc/h}$$

$$v_{NW} = 4{,}000 + 100 = 4{,}100 \text{ pc/h}$$

$$v = 4{,}100 + 900 = 5{,}000 \text{ pc/h}$$

$$VR = \frac{900}{5{,}000} = 0.180$$

Step 3: Determine Configuration Characteristics

The configuration is examined to determine the values of LC_{RF}, LC_{FR}, and N_{WL}. These determinations are illustrated in Exhibit 12-15. From these values, the minimum number of lane changes by weaving vehicles LC_{MIN} is then computed by using Equation 12-2.

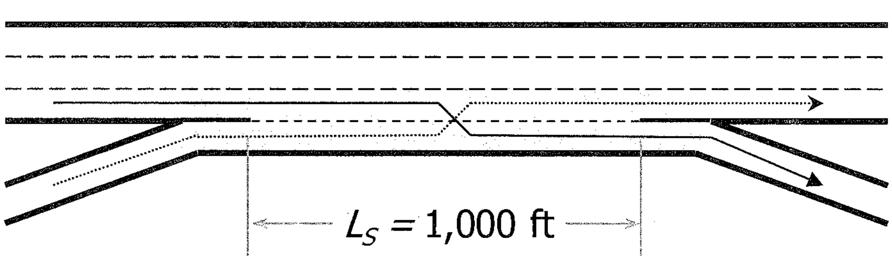


Exhibit 12-15
Configuration Characteristics for Example Problem 2

From Exhibit 12-15, it is clear that all ramp-to-freeway vehicles must make at least one lane change ($LC_{RF} = 1$), and all freeway-to-ramp vehicles must make at least one lane change ($LC_{FR} = 1$). It is also clear that a weaving maneuver can only be completed with a single lane change from the right lane of the freeway or the auxiliary lane ($N_{WL} = 2$). Then, by using Equation 12-2, LC_{MIN} is computed as

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

$$LC_{MIN} = (1 \times 600) + (1 \times 300) = 900 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length

The maximum length over which weaving operations may exist for the segment described is found by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.180)^{1.6}\right] - \left[1{,}566 \times 2\right] = 4{,}333 \text{ ft} > 1{,}000 \text{ ft}$$

As the maximum length for weaving operations significantly exceeds the actual length, this is a weaving segment, and the analysis continues.

Step 5: Determine Weaving Segment Capacity

The capacity of the weaving segment is controlled by one of two limiting factors: density reaches 43 pc/mi/ln or weaving demand reaches 2,400 pc/h for the configuration of Exhibit 12-15.

Capacity Limited by Density

The capacity limited by reaching a density of 43 pc/mi/ln is estimated by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765 L_s\right] + \left[119.8 N_{WL}\right]$$

$$c_{IWL} = 2{,}400 - \left[438.2(1+0.180)^{1.6}\right] + \left[0.0765 \times 1{,}000\right] + \left[119.8 \times 2\right]$$

$$c_{IWL} = 2{,}145 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 2{,}145 \times 4 \times 1 \times 1 = 8{,}580 \text{ pc/h}$$

Capacity Limited by Weaving Demand Flow

The capacity limited by the weaving demand flow is estimated by using Equation 12-7 and Equation 12-8:

$$c_{IW} = \frac{2{,}400}{VR} = \frac{2{,}400}{0.180} = 13{,}333 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} \times f_p = 13{,}333 \times 1 \times 1 = 13{,}333 \text{ pc/h}$$

The controlling capacity is the smaller value, or 8,580 pc/h. At this point, the value is usually stated as vehicles per hour. In this case, because inputs were already adjusted and were stated in passenger cars per hour, conversions back to vehicles per hour are not possible.

As the capacity is larger than the demand flow rate of 5,000 pc/h, LOS F does not exist, and the analysis continues.

Step 6: Determine Lane-Changing Rates

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}(N^2)(1 + ID)^{0.8}\right]$$

$$LC_W = 900 + 0.39\left[(1{,}000 - 300)^{0.5}4^2(1+1)^{0.8}\right] = 1{,}187 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{1{,}000 \times 1 \times 4{,}100}{10{,}000} = 410 < 1{,}300$$

$$LC_{NW} = (0.206 v_{NW}) + (0.542 L_S) - (192.6N)$$

$$LC_{NW} = (0.206 \times 4{,}100) + (0.542 \times 1{,}000) - (192.6 \times 4) = 616 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}187 + 616 = 1{,}803 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20:

$$W = 0.226\left(\frac{LC_{ALL}}{L_s}\right)^{0.789} = 0.226\left(\frac{1{,}803}{1{,}000}\right)^{0.789} = 0.360$$

Then

$$S_w = 15 + \left(\frac{FFS - 15}{1 + W}\right) = 15 + \left(\frac{75 - 15}{1 + 0.360}\right) = 59.1 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048\frac{v}{N}\right)$$

$$S_{NW} = 75 - (0.0072 \times 900) - \left(0.0048 \times \frac{5{,}000}{4}\right) = 62.5 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{4{,}100 + 900}{\left(\frac{4{,}100}{62.5}\right) + \left(\frac{900}{59.1}\right)} = 61.9 \text{ mi/h}$$

Step 8: Determine LOS

The average density in the weaving segment is estimated by using Equation 12-22.

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{5,000}{4}\right)}{61.9} = 20.2 \text{ pc/mi/ln}$$

From Exhibit 12-10, this density is within the stated boundaries of LOS C (20 to 28 pc/mi/ln). It is, however, very close to the LOS B boundary condition.

Discussion

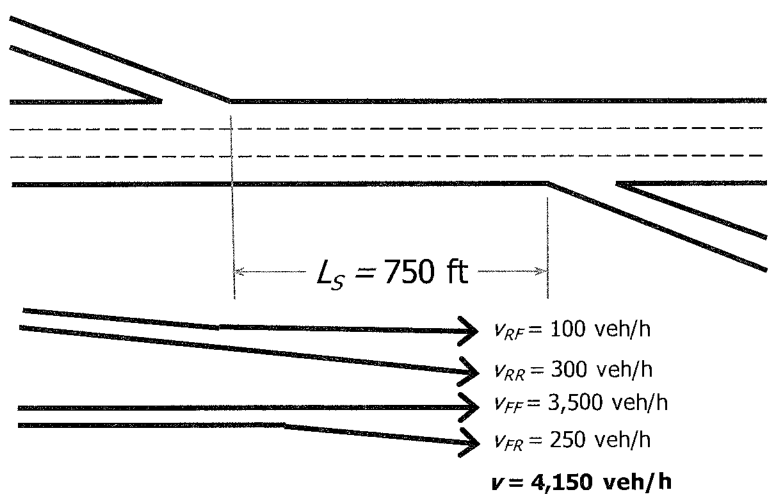
As noted, the segment is operating quite well (LOS C) and is very close to the LOS B boundary. Weaving and nonweaving speeds are relatively high, suggesting a stable flow. The demand flow rate of 5,000 pc/h is well below the capacity of the segment (8,580 pc/h). Weaving vehicles travel somewhat more slowly than nonweaving vehicles, which is typical of ramp-weave segments, where the vast majority of nonweaving vehicles are running from freeway to freeway.

EXAMPLE PROBLEM 3: LOS OF A TWO-SIDED WEAVING SEGMENT

The Weaving Segment

The weaving segment that is the subject of this example problem is shown in Exhibit 12-16.

Exhibit 12-16
Weaving Segment for Example Problem 3



What is the expected LOS and capacity for the weaving segment of Exhibit 12-16?

The Facts

In addition to the information contained in Exhibit 12-16, the following facts concerning the weaving segment are known:

PHF = 0.94 (all movements);

Heavy vehicles = 15% trucks, 0% RVs (all movements);

Driver population = regular commuters;

FFS = 60 mi/h;

c_{IFL} = 2,300 pc/h/ln (for FFS = 60 mi/h);

ID = 2 int/mi; and

Terrain = rolling.

Comments

Because this example illustrates the analysis of a two-sided weaving segment, several key parameters are redefined.

In a two-sided weaving segment, only the ramp-to-ramp flow is considered to be a weaving flow. While the freeway-to-freeway flow technically weaves with the ramp-to-ramp flow, the operation of freeway-to-freeway vehicles more closely resembles that of nonweaving vehicles. These vehicles generally make very few lane changes as they move through the segment in a freeway lane. This segment is in a busy urban corridor with a high interchange density and a relatively low FFS for the freeway.

Solution steps are the same as in the first two example problems. However, since the segment is a two-sided weaving segment, some of the key values will be computed differently as described in the methodology.

Component demand volumes will be converted to equivalent flow rates in passenger cars per hour under ideal conditions, and key demand parameters will be calculated. A maximum weaving length will be estimated to determine whether a weaving analysis is appropriate. The capacity of the weaving segment will be estimated to determine whether LOS F exists. If not, lane-changing parameters, speeds, density, and LOS will be estimated.

Step 1: Input Data

All information concerning this example problem is given in Exhibit 12-16 and the Facts section.

Step 2: Adjust Volume

To convert demand volumes to flow rates under equivalent ideal conditions, Chapter 11 must be consulted to obtain the following values:

E_T = 2.5 (for rolling terrain)

f_p = 1.0 (for regular commuters)

Then

$$f_{HV} = \frac{1}{1+P_T(E_T-1)} = \frac{1}{1+0.15(2.5-1)} = 0.816$$

Component demand volumes may now be converted to flow rates under equivalent ideal conditions:

$$v_{FF} = \frac{3{,}500}{0.94 \times 0.816 \times 1} = 4{,}561 \text{ pc/h}$$

$$v_{FR} = \frac{250}{0.94 \times 0.816 \times 1} = 326 \text{ pc/h}$$

$$v_{RF} = \frac{100}{0.94 \times 0.816 \times 1} = 130 \text{ pc/h}$$

$$v_{RR} = \frac{300}{0.94 \times 0.816 \times 1} = 391 \text{ pc/h}$$

Because this is a two-sided weaving segment, the only weaving flow is the ramp-to-ramp flow. All other flows are treated as nonweaving. Then

$$v_W = 391 \text{ pc/h}$$

$$v_{NW} = 4{,}561 + 326 + 130 = 5{,}017 \text{ pc/h}$$

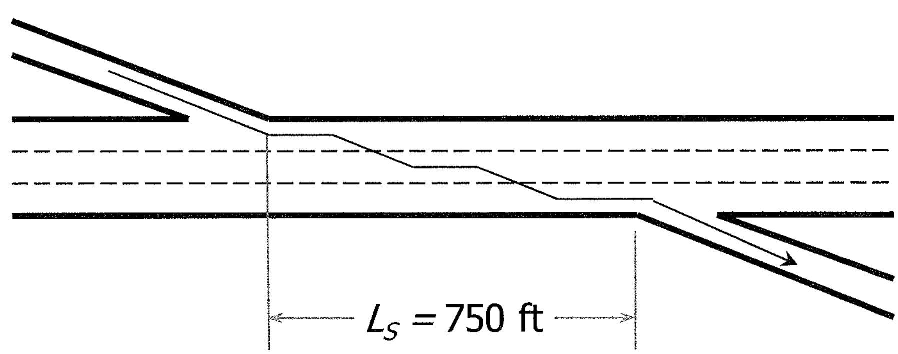
$$v = 5{,}017 + 391 = 5{,}408 \text{ pc/h}$$

$$VR = 391/5{,}408 = 0.072$$

Step 3: Determine Configuration Characteristics

The determination of configuration characteristics is also affected by the existence of a two-sided weaving segment. Exhibit 12-17 illustrates the determination of LC_{RR}, the key variable for two-sided weaving segments. For such segments, $N_{WL} = 0$ by definition.

Exhibit 12-17
Configuration Characteristics for Example Problem 3



From Exhibit 12-17, ramp-to-ramp vehicles must make two lane changes to complete their desired weaving maneuver. Then

$$LC_{MIN} = LC_{RR} \times v_{RR} = 2 \times 391 = 782 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length

The maximum length of a weaving segment for this configuration and demand scenario is estimated by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.072)^{1.6}\right] - \left[1{,}566 * 0\right] = 6{,}405 \text{ ft} > 750 \text{ ft}$$

In this two-sided configuration, the impacts of weaving on operations could be felt at lengths as long as 6,405 ft. As this is significantly greater than the actual length of 750 ft, this segment clearly operates as a weaving segment and, therefore, the methodology of this chapter should be applied.

Step 5: Determine Weaving Segment Capacity

The capacity of a two-sided weaving segment can only be estimated when a density of 43 pc/h/ln is reached. This estimation is made by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_s\right] + \left[119.8N_{WL}\right]$$

$$c_{IWL} = 2{,}300 - \left[438.2(1+0.072)^{1.6}\right] + \left[0.0765 \times 750\right] + \left[119.8 \times 0\right]$$

$$c_{IWL} = 1{,}867 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 1{,}867 \times 3 \times 0.816 \times 1 = 4{,}573 \text{ veh/h} > 4{,}150 \text{ veh/h}$$

Because the capacity of the segment exceeds the demand volume (in vehicles per hour), LOS F is not expected, and the analysis may be continued.

Step 6: Determine Lane-Changing Rates

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}(N^2)(1+ID)^{0.8}\right]$$

$$LC_W = 782 + 0.39\left[(750-300)^{0.5}3^2(1+2)^{0.8}\right] = 961 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{750 \times 2 \times 5{,}017}{10{,}000} = 753 < 1{,}300$$

$$LC_{NW} = (0.206v_{NW}) + (0.542L_s) - (192.6N)$$

$$LC_{NW} = (0.206 \times 5{,}017) + (0.542 \times 750) - (192.6 \times 3) = 862 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 961 + 862 = 1{,}823 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20:

$$W = 0.226\left(\frac{LC_{ALL}}{L_S}\right)^{0.789} = 0.226\left(\frac{1{,}823}{750}\right)^{0.789} = 0.456$$

Then

$$S_W = 15 + \left(\frac{FFS-15}{1+W}\right) = 15 + \left(\frac{60-15}{1+0.456}\right) = 45.9 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048 \frac{v}{N}\right)$$

$$S_{NW} = 75 - (0.0072 \times 782) - \left(0.0048 \times \frac{5{,}408}{3}\right) = 45.7 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{5{,}017 + 391}{\left(\frac{5{,}017}{45.7}\right) + \left(\frac{391}{45.9}\right)} = 45.7 \text{ mi/h}$$

Step 8: Determine LOS

The average density in this two-sided weaving segment is estimated by using Equation 12-22:

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{5{,}408}{3}\right)}{45.7} = 39.4 \text{ pc/mi/ln}$$

From Exhibit 12-10, this density is clearly in LOS E. It is not far from the 43 pc/h/ln that would likely cause a breakdown.

Discussion

This two-sided weaving segment operates at LOS E, not far from the LOS E/F boundary. The v/c ratio is 4,150/4,573 = 0.91. The major problem is that 300 veh/h crossing the freeway from ramp to ramp creates a great deal of turbulence in the traffic stream and limits capacity. Two-sided weaving segments do not operate well with such large numbers of ramp-to-ramp vehicles. If this were a basic freeway segment, the per lane flow rate of 5,408/3 = 1,803 pc/h/ln would not be considered excessive and would be well within a basic freeway segment's capacity of 2,300 pc/h/ln.

EXAMPLE PROBLEM 4: DESIGN OF A MAJOR WEAVING SEGMENT FOR A DESIRED LOS

The Weaving Segment

A weaving segment is to be designed between two major junctions in which two urban freeways join and then separate as shown in Exhibit 12-18. Entry and exit legs have the numbers of lanes shown. The maximum length of the weaving segment is 1,000 ft, based on the location of the junctions. The FFS of all entry and exit legs is 75 mi/h. All demands are shown as flow rates under equivalent ideal conditions.

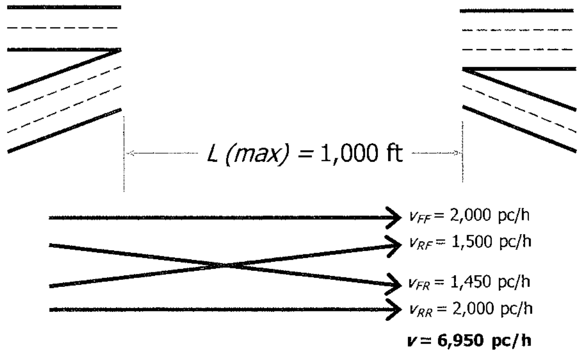


Exhibit 12-18
Weaving Segment for Example Problem 4

What design would be appropriate to deliver LOS C for the demand flow rates shown?

The Facts

In addition to the information contained in Exhibit 12-18, the following facts are known concerning this weaving segment:

PHF = 1.00 (all demands stated as flow rates);

Heavy vehicles = 0% trucks, 0% RVs (all demands in pc/h);

Driver population = regular commuters;

FFS = 75 mi/h;

c_{IFL} = 2,400 pc/h/ln (for FFS = 75 mi/h);

ID = 1 int/mi; and

Terrain = level.

Comments

As is the case in any weaving segment design, there are considerable constraints imposed. The problem states that the maximum length is 1,000 ft, no doubt limited by locational issues for the merge and diverge junctions. It is probably not worth investigating shorter lengths, and the maximum should be assumed for all trial designs. The simplest design merely connects entering lanes with exit lanes in a straightforward manner, producing a section of five lanes. A section with four lanes could be considered by merging two lanes into one at the entry gore and separating it into two again at the exit gore. In any event, the design is limited to a section of four or five lanes. No other widths would work without major additions to input and output legs. The configuration cannot be changed without adding a lane to at least one of the entry or exit legs. Thus, the initial trial will be at a length of 1,000 ft, with the five entry lanes connected directly to the five exit lanes, with no changes to the exit or entry leg designs. If this does not produce an acceptable operation, changes will be considered.

While the problem clearly states that all legs are freeways, no feasible configuration produces a two-sided weaving section. Thus, to fit within the one-sided analysis methodology, the right-side entry and exit legs will be classified as ramps in the computational analysis.

Step 1: Input Data – Trial 1

All input information is given in Exhibit 12-18 and in the accompanying Facts section for this example problem.

Step 2: Adjust Volume – Trial 1

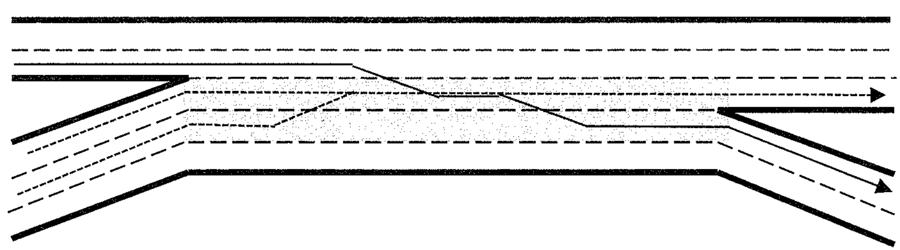
All demands are already stated as flow rates in passenger cars per hour under equivalent ideal conditions. No further adjustments are needed. Critical demand values are as follows:

$$v_{FF} = 2{,}000 \text{ pc/h}$$
$$v_{FR} = 1{,}450 \text{ pc/h}$$
$$v_{RF} = 1{,}500 \text{ pc/h}$$
$$v_{RR} = 2{,}000 \text{ pc/h}$$
$$v_W = 1{,}500 + 1{,}450 = 2{,}950 \text{ pc/h}$$
$$v_{NW} = 2{,}000 + 2{,}000 = 4{,}000 \text{ pc/h}$$
$$v = 2{,}950 + 4{,}000 = 6{,}950 \text{ pc/h}$$
$$VR = \frac{2{,}950}{6{,}950} = 0.424$$

Step 3: Determine Configuration Characteristics – Trial 1

Exhibit 12-19 illustrates the weaving segment formed under the assumed design discussed previously.

Exhibit 12-19
Trial Design 1
for Example Problem 4



The direct connection of entry and exit legs produces a weaving segment in which the ramp-to-freeway movement can be made without a lane change (LC_{RF} = 0). Freeway-to-ramp vehicles, however, must make two lane changes (LC_{FR} = 2). If the lane-changing pattern is considered, there are no lanes on the entering freeway leg from which a weaving maneuver can be made with one or no lane changes. Ramp drivers wishing to weave, however, can enter on either of the two left ramp lanes and weave with one or no lane changes. Thus, N_{WL} = 2.

By using Equation 12-2, LC_{MIN} is computed as

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$
$$LC_{MIN} = (0 \times 1{,}500) + (2 \times 1{,}450) = 2{,}900 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length – Trial 1

The maximum length of a weaving segment for this configuration and demand scenario is estimated by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.424)^{1.6}\right] - \left[1{,}566 \times 2\right] = 6{,}950 \text{ ft} > 1{,}000 \text{ ft}$$

As the maximum length is much greater than the actual length of 1,000 ft, it is appropriate to analyze the segment by using this chapter's methodology.

Step 5: Determine Weaving Segment Capacity – Trial 1

The capacity of the weaving segment is controlled by one of two limiting factors: density reaches 43 pc/mi/ln or weaving demand reaches 2,400 pc/h for the configuration of Exhibit 12-19.

Capacity Limited by Density

The capacity limited by reaching a density of 43 pc/mi/ln is estimated by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765 L_s\right] + \left[119.8 N_{WL}\right]$$

$$c_{IWL} = 2{,}400 - \left[438.2(1+0.424)^{1.6}\right] + \left[0.0765 \times 1{,}000\right] + \left[119.8 \times 2\right]$$

$$c_{IWL} = 1{,}944 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 1{,}944 \times 5 \times 1 \times 1 = 9{,}721 \text{ pc/h}$$

Capacity Limited by Weaving Demand Flow

The capacity limited by the weaving demand flow is estimated by using Equation 12-7 and Equation 12-8:

$$c_{IW} = \frac{2{,}400}{VR} = \frac{2{,}400}{0.424} = 5{,}654 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} \times f_p = 5{,}654 \times 1 \times 1 = 5{,}654 \text{ pc/h}$$

In this case, the capacity of the segment is limited by the maximum weaving flow rate of 5,654 pc/h, which is smaller than the total demand flow rate of 6,950 pc/h. Thus, this section is expected to operate at LOS F. No further analysis is possible with this methodology.

Discussion – Trial 1

This section would be expected to fail under the proposed design. The critical feature appears to be the configuration. Note that the capacity is limited by the maximum weaving flows that can be sustained, not by a density expected to produce queuing. This is primarily due to the freeway-to-ramp flow, which must make two lane changes. This number can be reduced to one by adding one lane to the "ramp" at the exit gore area. Not only does this reduce the number of lane changes made by 1,450 freeway-to-ramp vehicles, but it also increases the value of N_W from 2 to 3. In turn, this effectively increases the segment's capacity (as limited by weaving flow rate) to $3{,}500/VR = 3{,}500/0.424 = 8{,}255$ pc/h, which is well in excess of the demand flow rate of 6,950 pc/h. Another analysis (Trial 2) will be conducted by using this approach.

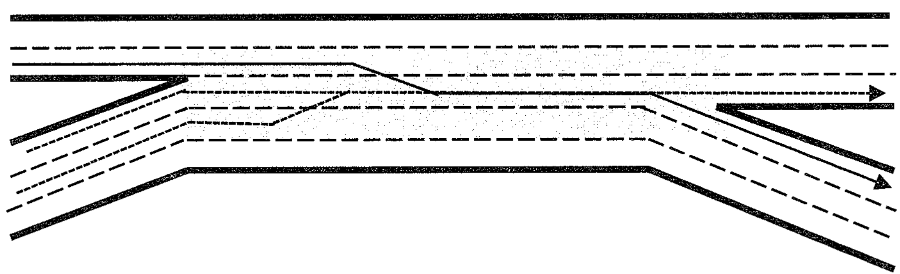
Steps 1 and 2: Input Data and Adjust Volume – Trial 2

Steps 1 and 2 are the same as for Trial 1. They are not repeated here. The new configuration affects the results beginning with Step 3.

Step 3: Determine Configuration Characteristics – Trial 2

Exhibit 12-20 illustrates the new configuration that will result from the changes discussed above. By adding a lane to the exit-ramp leg, the freeway-to-ramp movement can now be completed with only one lane change ($LC_{FR} = 1$). The value of LC_{RF} is not affected and remains 0. The right lane of the freeway-entry leg can also be used by freeway-to-ramp drivers to make a weaving maneuver with a single lane change, increasing N_{WL} to 3.

Exhibit 12-20
Trial Design 2
for Example Problem 4



Then

$$LC_{MIN} = (LC_{RF} \times v_{RF}) + (LC_{FR} \times v_{FR})$$

$$LC_{MIN} = (0 \times 1{,}500) + (1 \times 1{,}450) = 1{,}450 \text{ lc/h}$$

Step 4: Determine Maximum Weaving Length – Trial 2

The maximum length of a weaving segment for this configuration and demand scenario is estimated by using Equation 12-4:

$$L_{MAX} = \left[5{,}728(1+VR)^{1.6}\right] - \left[1{,}566 N_{WL}\right]$$

$$L_{MAX} = \left[5{,}728(1+0.424)^{1.6}\right] - \left[1{,}566 \times 3\right] = 5{,}391 \text{ ft} > 1{,}000 \text{ ft}$$

As the maximum length is much greater than the actual length of 1,000 ft, analyzing the segment by using this chapter's methodology is appropriate.

Step 5: Determine Weaving Segment Capacity – Trial 2

The capacity of the weaving segment is controlled by one of two limiting factors: density reaches 43 pc/mi/ln or weaving demand reaches 3,500 pc/h for the configuration of Exhibit 12-20.

Capacity Limited by Density

The capacity limited by reaching a density of 43 pc/mi/ln is estimated by using Equation 12-5 and Equation 12-6:

$$c_{IWL} = c_{IFL} - \left[438.2(1+VR)^{1.6}\right] + \left[0.0765L_s\right] + \left[119.8N_{WL}\right]$$

$$c_{IWL} = 2{,}400 - \left[438.2(1+0.424)^{1.6}\right] + \left[0.0765 \times 1{,}000\right] + \left[119.8 \times 3\right]$$

$$c_{IWL} = 2{,}064 \text{ pc/h/ln}$$

$$c_W = c_{IWL} \times N \times f_{HV} \times f_p = 2{,}064 \times 5 \times 1 \times 1 = 10{,}320 \text{ pc/h}$$

Capacity Limited by Weaving Demand Flow

The capacity limited by the weaving demand flow is estimated by using Equation 12-7 and Equation 12-8:

$$c_{IW} = \frac{3{,}500}{VR} = \frac{3{,}500}{0.424} = 8{,}255 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} \times f_p = 8{,}255 \times 1 \times 1 = 8{,}255 \text{ pc/h}$$

Once again, the capacity of the segment is limited by the maximum weaving flow rate: the difference is that now the capacity is 8,255 pc/h. This is larger than the total demand flow rate of 6,950 pc/h. Thus, this section is expected to operate without breakdown, and the analysis may continue.

Step 6: Determine Lane-Changing Rates – Trial 2

Equation 12-10 through Equation 12-15 are used to estimate the lane-changing rates of weaving and nonweaving vehicles in the weaving segment. In turn, these will be used to estimate weaving and nonweaving vehicle speeds.

Weaving Vehicle Lane-Changing Rate

$$LC_W = LC_{MIN} + 0.39\left[(L_s - 300)^{0.5}(N^2)(1+ID)^{0.8}\right]$$

$$LC_W = 1{,}450 + 0.39\left[(1{,}000 - 300)^{0.5} 5^2 (1+1)^{0.8}\right] = 1{,}899 \text{ lc/h}$$

Nonweaving Vehicle Lane-Changing Rate

$$I_{NW} = \frac{L_S ID v_{NW}}{10{,}000} = \frac{1{,}000 \times 1 \times 4{,}000}{10{,}000} = 400 < 1{,}300$$

$$LC_{NW} = (0.206v_{NW}) + (0.542L_S) - (192.6N)$$

$$LC_{NW} = (0.206 \times 4{,}000) + (0.542 \times 1{,}000) - (192.6 \times 5) = 403 \text{ lc/h}$$

Total Lane-Changing Rate

$$LC_{ALL} = LC_W + LC_{NW} = 1{,}899 + 403 = 2{,}302 \text{ lc/h}$$

Step 7: Determine Average Speeds of Weaving and Nonweaving Vehicles – Trial 2

The average speeds of weaving and nonweaving vehicles are computed from Equation 12-18 through Equation 12-20.

$$W = 0.226\left(\frac{LC_{ALL}}{L_s}\right)^{0.789} = 0.226\left(\frac{2{,}302}{1{,}000}\right)^{0.789} = 0.436$$

Then

$$S_W = 15 + \left(\frac{FFS - 15}{1 + W}\right) = 15 + \left(\frac{75 - 15}{1 + 0.436}\right) = 56.8 \text{ mi/h}$$

and

$$S_{NW} = FFS - (0.0072 LC_{MIN}) - \left(0.0048 \frac{v}{N}\right)$$

$$S_{NW} = 75 - (0.0072 \times 1{,}450) - \left(0.0048 \frac{6{,}950}{5}\right) = 57.9 \text{ mi/h}$$

Equation 12-21 is now used to compute the average speed of all vehicles in the segment:

$$S = \frac{v_{NW} + v_W}{\left(\frac{v_{NW}}{S_{NW}}\right) + \left(\frac{v_W}{S_W}\right)} = \frac{4{,}000 + 2{,}950}{\left(\frac{4{,}000}{57.9}\right) + \left(\frac{2{,}950}{56.8}\right)} = 57.4 \text{ mi/h}$$

Step 8: Determine the Level of Service – Trial 2

The average density in the weaving segment is estimated by using Equation 12-22:

$$D = \frac{\left(\frac{v}{N}\right)}{S} = \frac{\left(\frac{6{,}950}{5}\right)}{57.4} = 24.2 \text{ pc/mi/ln}$$

From Exhibit 12-10, this density is within the stated boundaries of LOS C (20 to 28 pc/mi/ln). As the design target was LOS C, the second trial design is acceptable.

Discussion – Trial 2

The relatively small change in the configuration makes all the difference in this design. LOS C can be achieved by adding a lane to the right exit leg; without it, the section fails due to excessive weaving turbulence. If the extra lane is not needed on the departing freeway leg, it would be dropped somewhere downstream, perhaps as part of the next interchange. The extra lane would have to be carried for several thousand feet to be effective. An added lane generally will not be fully utilized by drivers if they are aware that it will be immediately dropped.

EXAMPLE PROBLEM 5: CONSTRUCTING A SERVICE VOLUME TABLE FOR A WEAVING SEGMENT

This example shows how a table of service flow rates or service volumes or both can be constructed for a weaving section with certain specified characteristics. The methodology of this chapter does not directly yield service flow rates or service volumes, but they can be developed by using spreadsheets or more sophisticated computer programs.

The key issue is the definition of the threshold values for the various levels of service. For weaving sections on freeways, levels of service are defined as limiting densities as follows:

LOS	Maximum Density (pc/mi/ln)
A	10
B	20
C	28
D	35

By definition, the service flow rate at LOS E is the capacity of the weaving section, which may or may not be keyed to a density.

Before the construction of such a table is illustrated, several key definitions should be reviewed:

- *Service flow rate (under ideal conditions):* The maximum rate of flow under equivalent ideal conditions that can be sustained while maintaining the designated LOS (*SFI*, passenger cars per hour).
- *Service flow rate (under prevailing conditions):* The maximum rate of flow under prevailing conditions that can be sustained while maintaining the designated LOS (*SF*, vehicles per hour).
- *Service volume:* The maximum hourly volume under prevailing conditions that can be sustained while maintaining the designated LOS in the worst 15 min of the hour (*SV*, vehicles per hour).
- *Daily service volume:* The maximum AADT under prevailing conditions that can be sustained while maintaining the designated LOS in the worst 15 min of the peak hour (*DSV*, vehicles per day).

Note that flow rates are for a 15-min period, often a peak 15 min within the analysis hour, or the peak hour. These values are related as follows:

$$SF_i = SFI_i \times f_{HV} \times f_p$$

$$SV_i = SF_i \times PHF$$

$$DSV_i = \frac{SV_i}{KD}$$

This chapter's methodology estimates both the capacity and the density expected in a weaving segment of given geometric and demand characteristics. Conceptually, the approach to generating values of *SFI* is straightforward: for any given situation, keep increasing the input flow rates until the boundary density for the LOS is reached; the input flow rate is the *SFI* for that situation and LOS. This obviously involves many iterations. A spreadsheet can be programmed to do this, either semiautomatically with manual input of demands, or fully automatically, with the spreadsheet automatically generating solutions until a density match is found. The latter method is not very efficient and involves a typical spreadsheet program running for several hours. A program could, of course, be written to automate the entire process.

An Example

While all of the computations cannot be shown, demonstration results for a specific case can be illustrated. A service volume table is desired for a weaving section with the following characteristics:

- One-sided major weaving section
- Demand splits as follows:
  - v_{FF} = 65% of v
  - v_{RF} = 15% of v
  - v_{FR} = 12% of v
  - v_{RR} = 8% of v
- Trucks = 10%, RVs = 0%
- Level terrain
- PHF = 0.93
- f_p = 1.00
- ID = 1 int/mi
- FFS = 65 mi/h

For these characteristics, a service volume table can be constructed for a range of lengths and widths and for configurations in which N_W is 2 and 3. For illustrative purposes, lengths of 500, 1,000, 1,500, 2,000, and 2,500 ft and widths of three, four, or five lanes will be used. In a major weaving section, one weaving flow does not have to make a lane change. For the purposes of this example, it is assumed that the ramp-to-freeway movement has this characteristic. The freeway-to-ramp movement would require one or two lane changes, on the basis of the value of N_{WL}.

First Computations

Initial computations will be aimed at establishing values of *SFI* for the situations described. A spreadsheet will be constructed in which the first column is the flow rate to be tested (in passenger cars per hour under ideal conditions), and the last column produces a density. Each line will be iterated (manually in this case) until each threshold density value is reached. Intermediate columns will be programmed to produce the intermediate results needed to get to this result. Because maximum length and capacity are decided at intermediate points, the applicable results will be manually entered before continuing. Such a procedure is less difficult than it seems once the basic computations are programmed. Manual iteration using the input flow rate is very efficient, as the operator will observe how fast the results are converging to the desired threshold and will change the inputs accordingly.

The results of a first computation are shown in Exhibit 12-21. They represent service flow rates under ideal conditions, *SFI*. Consistent with the HCM's results presentation guidelines (Chapter 7, Interpreting HCM and Alternative Tool Results), all hourly service flow rates and volumes in the following exhibits have been rounded down to the nearest 100 passenger cars or vehicles for presentation.

Exhibit 12-21
Service Flow Rates Under Ideal Conditions (*SFI*) for Example Problem 5 (pc/h)

LOS	500	1,000	1,500	2,000	2,500	500	1,000	1,500	2,000	2,500
	Length of Weaving Section (ft)									
	$N = 3$; $N_{WL} = 2$					**$N = 3$; $N_{WL} = 3$**				
A	1,700	1,700	1,700	1,700	1,700	1,800	1,800	1,800	1,800	1,800
B	3,200	3,200	3,200	3,200	3,200	3,300	3,300	3,400	3,400	3,400
C	4,200	4,200	4,300	4,300	4,300	4,400	4,500	4,500	4,500	4,500
D	5,000	5,100	5,100	5,100	5,100	5,300	5,400	5,400	5,500	5,500
E	5,900	6,000	6,100	6,300	6,400	6,300	6,400	6,500	6,600	6,700
	$N = 4$; $N_{WL} = 2$					**$N = 4$; $N_{WL} = 3$**				
A	2,200	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300
B	4,100	4,200	4,200	4,200	4,200	4,300	4,400	4,400	4,400	4,400
C	5,400	5,500	5,500	5,500	5,600	5,800	5,900	5,900	5,900	5,900
D	6,300	6,500	6,500	6,600	6,600	6,900	7,000	7,100	7,100	7,100
E	7,900	8,000	8,200	8,400	8,500	8,400	8,500	8,700	8,800	9,000
	$N = 5$; $N_{WL} = 2$					**$N = 5$; $N_{WL} = 3$**				
A	2,800	2,800	2,800	2,800	2,800	2,900	2,900	2,900	2,900	2,900
B	5,000	5,100	5,100	5,100	5,100	5,400	5,400	5,400	5,500	5,500
C	6,500	6,600	6,700	6,700	6,700	7,100	7,200	7,200	7,300	7,300
D	7,600	7,800	7,900	7,900	7,900	8,400	8,600	8,700	8,700	8,700
E	8,800	8,800	8,800	8,800	8,800	10,500	10,700	10,900	11,100	11,200

Exhibit 12-22 shows service flow rates under prevailing conditions, *SF*. Each value in Exhibit 12-21 (before rounding) is multiplied by

$$f_{HV} = \frac{1}{1+0.10(1.5-1)} = 0.952$$

$$f_p = 1.00$$

Exhibit 12-22
Service Flow Rates Under Prevailing Conditions (*SF*) for Example Problem 5 (veh/h)

LOS	500	1,000	1,500	2,000	2,500	500	1,000	1,500	2,000	2,500
	Length of Weaving Section (ft)									
	$N = 3$; $N_{WL} = 2$					**$N = 3$; $N_{WL} = 3$**				
A	1,600	1,600	1,600	1,600	1,600	1,700	1,700	1,700	1,700	1,700
B	3,000	3,000	3,100	3,100	3,100	3,100	3,200	3,200	3,200	3,200
C	4,000	4,000	4,100	4,100	4,100	4,200	4,300	4,300	4,300	4,300
D	4,700	4,800	4,900	4,900	4,900	5,100	5,100	5,200	5,200	5,200
E	5,600	5,700	5,800	5,900	6,100	6,000	6,100	6,200	6,200	6,400
	$N = 4$; $N_{WL} = 2$					**$N = 4$; $N_{WL} = 3$**				
A	2,100	2,100	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200
B	3,900	4,000	4,000	4,000	4,000	4,100	4,200	4,200	4,200	4,200
C	5,100	5,200	5,200	5,300	5,300	5,500	5,600	5,600	5,600	5,600
D	5,900	6,200	6,200	6,300	6,300	6,600	6,700	6,700	6,800	6,800
E	7,500	7,700	7,800	7,900	8,100	8,000	8,100	8,200	8,400	8,500
	$N = 5$; $N_{WL} = 2$					**$N = 5$; $N_{WL} = 3$**				
A	2,600	2,700	2,700	2,700	2,700	2,700	2,700	2,800	2,800	2,800
B	4,700	4,800	4,900	4,900	4,900	5,100	5,100	5,200	5,200	5,200
C	6,200	6,300	6,300	6,400	6,400	6,700	6,800	6,900	6,900	6,900
D	7,300	7,400	7,500	7,500	7,500	8,000	8,200	8,200	8,300	8,300
E	8,400	8,400	8,400	8,400	8,400	10,000	10,200	10,300	10,500	10,700

Exhibit 12-23 shows service volumes, *SV*. Each value in Exhibit 12-22 (before rounding) is multiplied by a PHF of 0.93.

Exhibit 12-23
Service Volumes Under Prevailing Conditions (*SV*) for Example Problem 5 (veh/h)

	Length of Weaving Section (ft)									
LOS	**500**	**1,000**	**1,500**	**2,000**	**2,500**	**500**	**1,000**	**1,500**	**2,000**	**2,500**
	$N = 3$; $N_{WL} = 2$					$N = 3$; $N_{WL} = 3$				
A	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500	1,500
B	2,800	2,800	2,800	2,800	2,900	2,900	2,900	3,000	3,000	3,000
C	3,700	3,700	3,800	3,800	3,800	3,900	4,000	4,000	4,000	4,000
D	4,400	4,500	4,500	4,500	4,500	4,700	4,800	4,800	4,800	4,800
E	5,200	5,300	5,400	5,500	5,600	5,500	5,600	5,700	5,800	5,900
	$N = 4$; $N_{WL} = 2$					$N = 4$; $N_{WL} = 3$				
A	2,000	2,000	2,000	2,000	2,000	2,000	2,100	2,100	2,100	2,100
B	3,600	3,700	3,700	3,700	3,700	3,800	3,900	3,900	3,900	3,900
C	4,700	4,800	4,900	4,900	4,900	5,100	5,200	5,200	5,200	5,200
D	5,500	5,700	5,800	5,800	5,800	6,100	6,200	6,300	6,300	6,300
E	7,000	7,100	7,300	7,400	7,500	7,400	7,500	7,700	7,800	7,900
	$N = 5$; $N_{WL} = 2$					$N = 5$; $N_{WL} = 3$				
A	2,400	2,500	2,500	2,500	2,500	2,500	2,500	2,600	2,600	2,600
B	4,400	4,500	4,500	4,500	4,500	4,700	4,800	4,800	4,800	4,800
C	5,700	5,800	5,900	5,900	5,900	6,200	6,400	6,400	6,400	6,400
D	6,700	6,900	7,000	7,000	7,000	7,500	7,600	7,700	7,700	7,700
E	7,800	7,800	7,800	7,800	7,800	9,300	9,400	9,600	9,800	9,900

Exhibit 12-24 shows daily service volumes, *DSV*. An illustrative K-factor of 0.08 (typical of a large urban area) and an illustrative D-factor of 0.55 (typical of an urban route without strong peaking by direction) are used. Each (nonrounded) value used to generate Exhibit 12-23 was divided by both of these numbers.

Exhibit 12-24
Daily Service Volumes Under Prevailing Conditions (*DSV*) for Example Problem 5 (veh/day)

	Length of Weaving Section (ft)									
LOS	**500**	**1,000**	**1,500**	**2,000**	**2,500**	**500**	**1,000**	**1,500**	**2,000**	**2,500**
	$N = 3$; $N_{WL} = 2$					$N = 3$; $N_{WL} = 3$				
A	35,200	35,200	35,400	35,500	35,600	36,200	36,300	36,300	36,300	36,300
B	64,300	65,300	65,500	65,700	66,100	67,600	68,000	68,400	68,400	68,400
C	84,700	86,100	86,700	87,200	87,500	89,700	90,900	91,500	91,700	91,900
D	100,800	102,800	103,600	104,000	104,400	107,800	109,600	110,200	110,600	110,800
E	119,800	122,100	124,400	126,700	129,100	127,000	129,400	131,600	132,800	136,300
	$N = 4$; $N_{WL} = 2$					$N = 4$; $N_{WL} = 3$				
A	45,800	46,200	46,600	46,600	46,600	47,600	47,800	47,800	47,900	47,900
B	83,300	84,700	85,100	85,500	85,700	88,300	89,300	89,500	89,700	89,900
C	108,600	110,800	111,600	112,200	112,600	117,100	118,700	119,500	120,100	120,300
D	126,700	131,300	132,400	133,200	133,600	140,000	142,400	143,600	144,000	144,400
E	159,800	162,800	165,900	169,000	172,100	169,400	172,500	175,400	178,600	181,700
	$N = 5$; $N_{WL} = 2$					$N = 5$; $N_{WL} = 3$				
A	56,300	57,100	57,300	57,500	57,500	58,700	58,900	59,300	59,400	59,400
B	101,400	103,000	103,600	104,200	104,400	108,600	109,600	110,000	110,600	110,800
C	131,300	133,800	135,000	135,800	136,200	142,800	145,400	146,200	146,800	147,400
D	154,500	157,700	159,100	159,900	160,300	170,600	173,600	175,000	175,800	175,800
E	178,800	178,800	178,800	178,800	178,800	211,800	215,600	219,500	223,300	227,200

6. REFERENCES

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

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CHAPTER 13
FREEWAY MERGE AND DIVERGE SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

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—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 **Chapter 13, Freeway Merge and Diverge Segments,** 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.

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. When a ramp connection to a surface facility (such as a multilane highway) or a C-D roadway is designed for high-speed merging or diverging without control, it 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; rather, it is discussed in Chapter 22, Interchange Ramp Terminals. 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. Mainline operations can also be affected by platoon entries created by ramp–street intersection control.

See Chapter 22 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, proximity of other ramps, and other elements all affect merging and diverging operations.

CLASSIFICATION OF RAMPS

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. Some, however, join with the left lane(s) of the freeway and are classified as left-hand ramps.
- 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 Special Cases 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. Two-lane off-ramp roadways, however, often have two lanes at the diverge point as well. These are treated by using a special model (see this chapter's Special Cases section).
- Ramp–freeway merge and diverge operations are affected by the size of the freeway segment (in one direction).
- Ramp–freeway merge and diverge operations may be affected by the proximity of adjacent ramps and the flow rates on those ramps.

Left-hand ramps are considered as special cases later in this chapter.

Merge and diverge segments with two lanes at the point of merge or diverge are considered as special cases later in this chapter.

The number of combinations of these characteristics that can occur is very 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

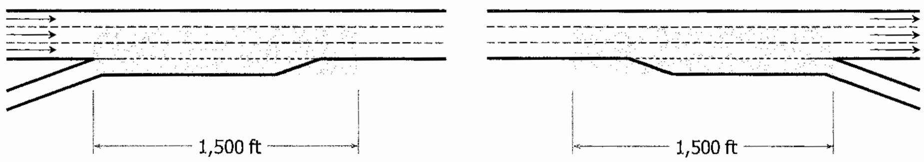
Ramps and ramp junctions do not operate independently of the roadways they connect. Thus, operating conditions on the main roadways can affect operations on the ramp and ramp junctions, and vice versa. In particular, a breakdown [Level of Service (LOS) F] at a ramp–freeway junction may have serious effects on the freeway upstream or downstream of the junction. These effects can influence freeway operations for miles in the worst cases.

With undersaturated conditions, the operational impacts of ramp–freeway junctions occur within a 1,500-ft-long influence area.

For most stable operations, however, 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 13-1 illustrates the definition of ramp influence areas. For left-hand ramps, the two leftmost lanes of the freeway are affected.

The influence area includes the acceleration/deceleration lane and the right two lanes of the freeway (left two lanes for left-hand ramps).

Exhibit 13-1
Ramp Influence Areas Illustrated



(a) Merge Influence Area (b) Diverge Influence Area

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.

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.

Ramp influence areas experience higher rates of lane-changing than normally occur in basic freeway segments.

At off-ramps, the basic maneuver is a diverge—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 act to limit ramp flow, causing diversion to other interchanges or routes.

Exhibit 13-1 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 Special Cases section of this chapter.

BASE CONDITIONS

The base conditions for the methodology presented in this chapter are the same as for other types of freeway segments:

Base conditions for merge and diverge segments are the same as for other types of freeway segments.

- No heavy vehicles,
- 12-ft lanes,
- Adequate lateral clearances (≥6 ft), and
- Road users familiar with the facility (i.e., $f_p = 1.00$).

LOS CRITERIA FOR MERGE AND DIVERGE SEGMENTS

LOS A–E is 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 where the off-ramp demand exceeds the off-ramp capacity.

At LOS A, unrestricted operations exist, and the density is low enough to permit smooth merging or diverging with very 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.

Exhibit 13-2 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 22, Interchange Ramp Terminals.

Exhibit 13-2
LOS Criteria for Freeway Merge and Diverge Segments

LOS	Density (pc/mi/ln)	Comments
A	≤10	Unrestricted operations
B	>10–20	Merging and diverging maneuvers noticeable to drivers
C	>20–28	Influence area speeds begin to decline
D	>28–35	Influence area turbulence becomes intrusive
E	>35	Turbulence felt by virtually all drivers
F	Demand exceeds capacity	Ramp and freeway queues form

REQUIRED INPUT DATA

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.

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, rolling, or mountainous; or percent grade and length;
4. Heavy vehicle presence: percent trucks and buses, percent recreational vehicles (RVs);
5. Demand flow rate immediately upstream of the ramp–freeway junction;
6. Peak hour factor: up to 1.00; and
7. Driver population factor: 0.85–1.00.

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, one may be estimated by using the methodology for basic freeway segments presented in Chapter 11, Basic Freeway 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 14, Multilane Highways, 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 factor is generally 1.00, unless the demand 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.

Data Describing the Ramp–Freeway Junction

The following information concerning the ramp–freeway junction is needed to conduct an analysis:

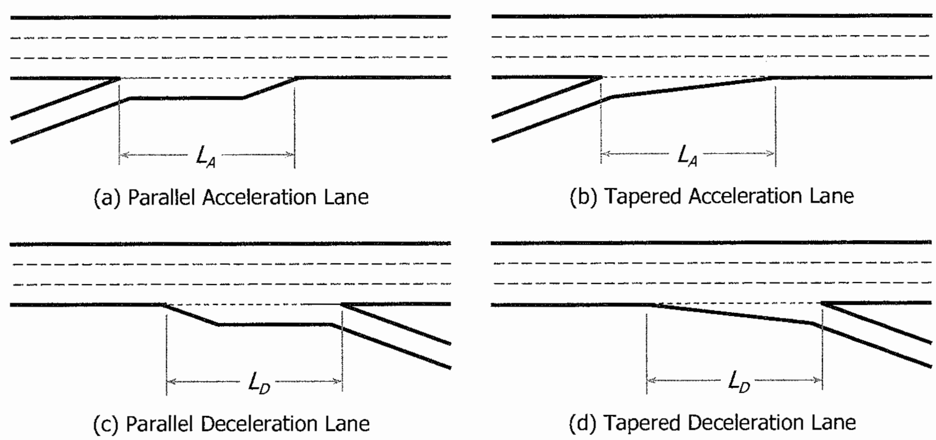
1. Type of ramp: on-ramp, off-ramp, major merge, major diverge;
2. Side of junction: right-hand, left-hand;
3. Number of lanes on ramp roadway: 1 lane, 2 lanes;
4. Number of ramp lanes at ramp–freeway junction: 1 lane, 2 lanes;
5. Length of acceleration/deceleration lane(s);
6. FFS of the ramp roadway: 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, percent RVs;
10. Peak hour factor: up to 1.00;
11. Driver population factor: 0.85–1.00; and
12. For adjacent upstream or downstream ramps:
    a. Upstream or downstream distance to the merge/diverge under study,
    b. Demand flow rate on the upstream or downstream ramp, and
    c. Peak hour factor 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 13-3 illustrates lengths for both parallel and tapered ramp designs.

Exhibit 13-3
Measuring the Length of Acceleration and Deceleration Lanes



Source: *Traffic Engineering,* 3rd edition (*2*).

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.

2. 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 (LOS F) and to analyze undersaturated operations (LOS A–E) at ramp–freeway junctions. Chapter 10, Freeway Facilities, 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 3-37 (*1*, *2*). Some special applications resulted from adaptations of procedures developed in the 1970s (*3*). American Association of State Highway and Transportation Officials policies (*4*) contain additional material on the geometric design and design criteria for ramps.

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

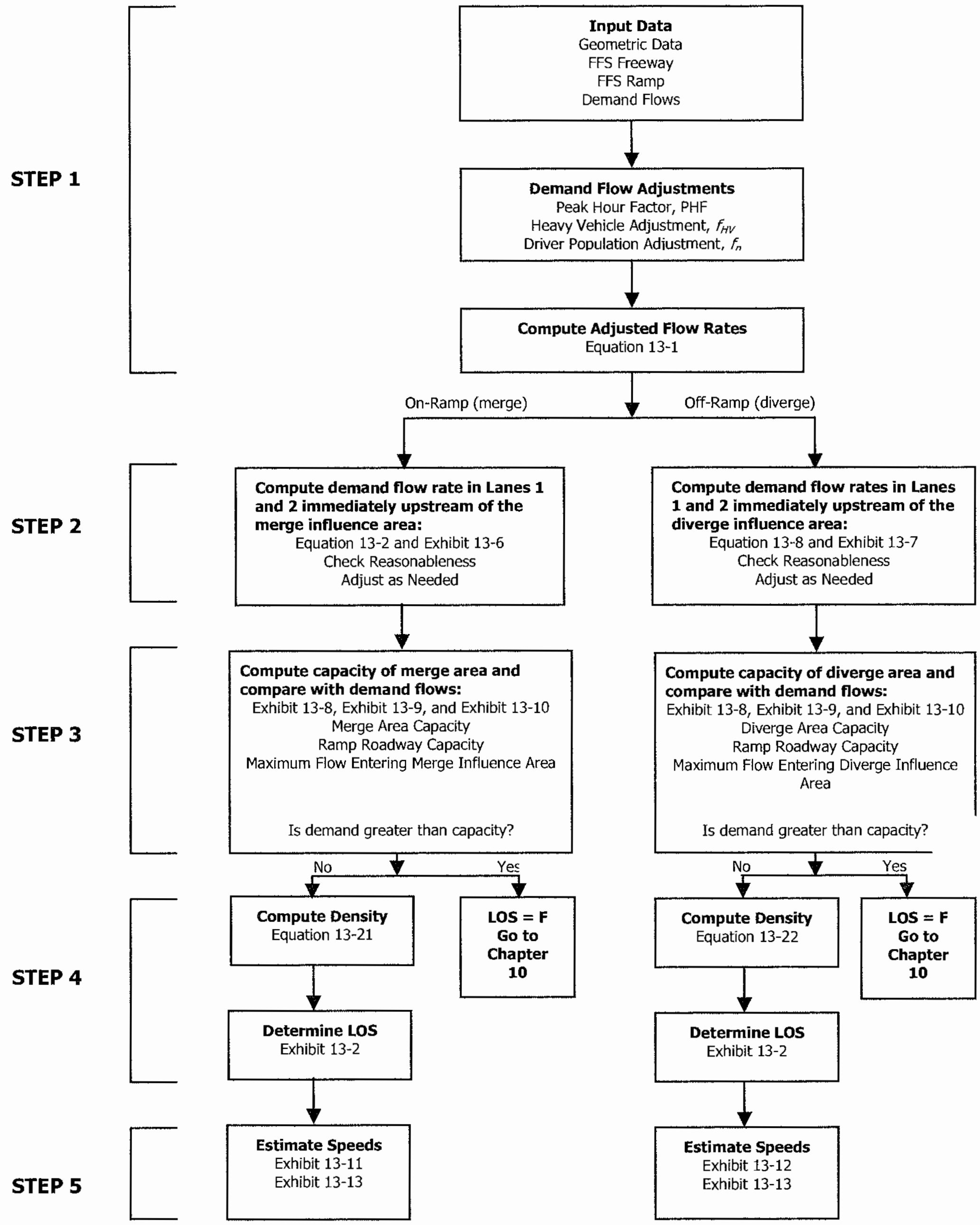
- Special lanes, such as high-occupancy vehicle (HOV) lanes, as ramp entry lanes;
- Ramp metering; or
- Intelligent transportation system features.

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.

OVERVIEW

Exhibit 13-4 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.

Exhibit 13-4
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 13-1. 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 13-5 illustrates key variables involved in the methodology.

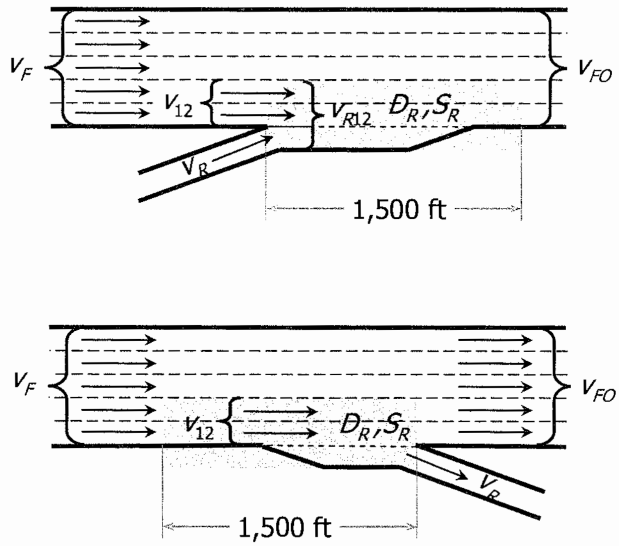


Exhibit 13-5
Key Ramp Junction Variables

The variables illustrated in Exhibit 13-5 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).

The computational process illustrated in Exhibit 13-4 may be broken 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.

COMPUTATIONAL STEPS

The methodology was calibrated for one-lane, right-side ramp–freeway junctions. Other situations are addressed in the Special Cases section.

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 the Special Cases 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 13-1

$$v_i = \frac{V_i}{PHF \times f_{HV} \times f_p}$$

where

v_i = demand flow rate for movement i (pc/h),

V_i = demand volume for movement i (veh/h),

PHF = peak hour factor,

f_{HV} = adjustment factor for heavy vehicle presence, and

f_p = adjustment factor for driver population.

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 11, Basic Freeway 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. An on-ramp, for example, 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 size 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 trivial: 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 as "adjacent" ramps under these procedures.

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 13-2:

Equation 13-2

$$v_{12} = v_F \times P_{FM}$$

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.

Exhibit 13-6 shows the algorithms used to determine P_{FM} for on-ramps or merge areas. All variables in Exhibit 13-6 are as previously defined.

Three equations are provided for six-lane freeways. Equation 13-3 is the base case for isolated ramps and for cases in which adjacent ramps are not found to influence merging operations. Equation 13-4 addresses cases with an upstream adjacent off-ramp, while Equation 13-5 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 13-3 is applied in such cases.

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. Data have been insufficient to determine whether adjacent ramps influence lane distribution on four-lane (one direction) freeway segments, and thus no such impact is used in 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 13-3 should be used. If the actual distance is shorter than L_{EQ}, then Equation 13-4 or Equation 13-5 should be used as appropriate.

Exhibit 13-6
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	$P_{FM} = 0.5775 + 0.000028\,L_A$ (Equation 13-3) $P_{FM} = 0.7289 - 0.0000135\,(v_F + v_R) - 0.003296 S_{FR} + 0.000063 L_{UP}$ (Equation 13-4) $P_{FM} = 0.5487 + 0.2628\,(v_D / L_{DOWN})$ (Equation 13-5)
8	For $v_F / S_{FR} \le 72$: $P_{FM} = 0.2178 - 0.000125 v_R + 0.01115\,(L_A / S_{FR})$ For $v_F / S_{FR} > 72$: $P_{FM} = 0.2178 - 0.000125 v_R$

SELECTING EQUATIONS FOR P_{FM} FOR SIX-LANE FREEWAYS

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	On	None	Equation 13-3
None	On	On	Equation 13-3
None	On	Off	Equation 13-5 or 13-3
On	On	None	Equation 13-3
Off	On	None	Equation 13-4 or 13-3
On	On	On	Equation 13-3
On	On	Off	Equation 13-5 or 13-3
Off	On	On	Equation 13-4 or 13-3
Off	On	Off	Equation 13-5 or 13-4 or 13-3

Note: [a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
If an adjacent diverge on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 13-3.

The equilibrium distance is obtained by finding the distance at which Equation 13-3 would yield the same value of P_{FM} as Equation 13-4 or Equation 13-5, as appropriate. This results in the following:

For adjacent upstream off-ramps, use Equation 13-6:

$$L_{EQ} = 0.214(v_F + v_R) + 0.444 L_A + 52.32 S_{FR} - 2{,}403$$

Equation 13-6

For adjacent downstream off-ramps, use Equation 13-7:

$$L_{EQ} = \frac{v_D}{0.1096 + 0.000107 L_A}$$

Equation 13-7

where all terms are as previously defined.

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 both adjacent upstream and downstream off-ramps are present, the larger resulting value of P_{FM} is used.

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, or where they are on the left side of the freeway, Equation 13-3 is always applied.

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 13-3.

Estimating Flow in Lanes 1 and 2 for Off-Ramps (Diverge Areas)

When approaching an off-ramp (diverge area), all off-ramp traffic 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 13-8:

Equation 13-8

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

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), and

P_{FD} = proportion of diverging traffic remaining in Lanes 1 and 2 immediately upstream of the deceleration lane.

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 13-7 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 trivial, since only Lanes 1 and 2 exist.

Exhibit 13-7
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	**Equation 13-9** $P_{FD} = 0.760 - 0.000025v_F - 0.000046v_R$ **Equation 13-10** $P_{FD} = 0.717 - 0.000039v_F + 0.604(v_U / L_{UP})$ **Equation 13-11** $P_{FD} = 0.616 - 0.000021v_F + 0.124(v_D / L_{DOWN})$
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 13-9
None	Off	On	Equation 13-9
None	Off	Off	Equation 13-11 or 13-9
On	Off	None	Equation 13-10 or 13-9
Off	Off	None	Equation 13-9
On	Off	On	Equation 13-10 or 13-9
On	Off	Off	Equation 13-11, 13-10, or 13-9
Off	Off	On	Equation 13-9
Off	Off	Off	Equation 13-11 or 13-9

Note: [a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
If an adjacent ramp on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 13-9.

For six-lane freeways, three equations are presented. Equation 13-9 is the base case for isolated ramps or for cases in which the impact of adjacent ramps can be ignored. Equation 13-10 addresses cases in which there is an adjacent upstream on-ramp, while Equation 13-11 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 13-7 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 13-9 yields the same value of P_{FD} as Equation 13-10 (for adjacent upstream on-ramps) or Equation 13-11 (adjacent downstream off-ramps). When the actual distance between ramps is greater than or equal to L_{EQ}, Equation 13-9 is used. When the actual distance between ramps is less than L_{EQ}, Equation 13-10 or Equation 13-11 is used as appropriate.

For adjacent upstream on-ramps, use Equation 13-12 to find the equilibrium distance:

$$L_{EQ} = \frac{v_u}{0.071 + 0.000023 v_F - 0.000076 v_R}$$

Equation 13-12

For adjacent downstream off-ramps, use Equation 13-13:

$$L_{EQ} = \frac{v_D}{1.15 - 0.000032 v_F - 0.000369 v_R}$$

Equation 13-13

where all terms are as previously defined.

A special case exists when both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present. In such cases, 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 both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present, the larger resulting value of P_{FD} is used.

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 or major diverge configurations, or where they are on the left side of the freeway, Equation 13-9 is always applied.

When an adjacent ramp to a diverge area on a six-lane freeway is not a one-lane, right-side ramp, apply Equation 13-9.

Checking the Reasonableness of the Lane Distribution Prediction

The algorithms of Exhibit 13-6 and Exhibit 13-7 were developed through regression analysis of a large database. Unfortunately, regression-based models may yield unreasonable or unexpected results when applied outside the strict limits of the calibration database, and they may have inconsistencies at their boundaries.

Therefore, it is necessary to apply some limits to predicted values of flow in Lanes 1 and 2 (v_{12}). The following limitations apply to all such predictions:

Reasonableness checks on the value of v_{12}.

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.

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 13-14:

Equation 13-14

$$v_3 = v_F - v_{12}$$

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, use Equation 13-15:

Equation 13-15

$$v_{12a} = v_F - 2{,}700$$

If v_3 is greater than 1.5 × (v_{12}/2), use Equation 13-16:

Equation 13-16

$$v_{12a} = \left(\frac{v_F}{1.75}\right)$$

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 13-17:

Equation 13-17

$$v_{av34} = \frac{v_F - v_{12}}{2}$$

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 13-18:

Equation 13-18

$$v_{12a} = v_F - 5{,}400$$

If v_{av34} is greater than 1.5 × (v_{12}/2), use Equation 13-19:

Equation 13-19

$$v_{12a} = \left(\frac{v_F}{2.50}\right)$$

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 capacity 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 diminish the capacity of the freeway.

Failure of a diverge junction is usually caused by a capacity deficiency at the ramp–street terminal or on the off-ramp roadway.

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.

While this methodology establishes a maximum desirable rate of flow entering the ramp influence area, exceeding this value does not cause a failure. 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}. At on-ramps, however, 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 13-20:

Equation 13-20

$$v_{R12} = v_{12} + v_R$$

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 13-8 shows capacity values for ramp–freeway junctions. Exhibit 13-9 shows similar values for high-speed ramps on multilane highways and C-D roadways within freeway interchanges. Exhibit 13-10 shows the capacity of ramp roadways.

Exhibit 13-8
Capacity of Ramp–Freeway Junctions (pc/h)

FFS (mi/h)	Capacity of Upstream/Downstream Freeway Segment[a] — No. of Lanes in One Direction: 2	3	4	>4	Max. Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Max. Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
≥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 13-9
Capacity of High-Speed Ramp Junctions on Multilane Highways and C-D Roadways (pc/h)

FFS (mi/h)	Capacity of Upstream/Downstream Highway or C-D Segment[a] — No. of Lanes in One Direction: 2	3	>3	Max. Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Max. Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
≥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 13-10
Capacity of Ramp Roadways (pc/h)

Ramp FFS S_{FR} (mi/h)	Capacity of Ramp Roadway: 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

Note: 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 13-8 and Exhibit 13-9.

Ramp–Freeway Junction Capacity Checkpoint

As noted previously, it is generally the capacity of the upstream or downstream freeway segment that limits flow through a merge or diverge area, assuming that the number of freeway lanes entering and leaving the ramp junction is the same. In such cases, the critical checkpoint for freeway capacity is

- 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 downstream of the ramp influence area.

Failure of any ramp–freeway junction capacity check results in LOS F.

Failure of any ramp–freeway junction capacity check (i.e., demand exceeds capacity: v/c is greater than 1.00) results in LOS F.

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 19, 20, or 21) or signalized interchange ramp terminals (Chapter 22).

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

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.

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.

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 13-21:

Equation 13-21

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

where D_R is the density in the ramp influence area (pc/mi/ln) and all other variables are as previously defined.

The equation is logical. 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 physical 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 13-22:

Equation 13-22

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

where all variables are as previously defined.

This equation also follows logical trends. 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 13-21 or Equation 13-22. Exhibit 13-2, shown previously, 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 it is determined that a merge or diverge segment is operating (or expected to operate at) LOS F, the analyst should go to Chapter 10, 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 in 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 (Chapter 10), while the first type of speed provides a useful companion measure to density within the ramp influence area in all cases.

Exhibit 13-11 and Exhibit 13-12 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 13-13 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 13-11
Estimating Speed at On-Ramp (Merge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS - (FFS - 42)M_S$	
	$M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002\ (L_A S_{FR}/1{,}000)$	
Outer lanes of freeway	$S_O = FFS$	$v_{OA} < 500$ pc/h
	$S_O = FFS - 0.0036(v_{OA} - 500)$	500 pc/h $\le v_{OA} \le$ 2,300 pc/h
	$S_O = FFS - 6.53 - 0.006(v_{OA} - 2{,}300)$	$v_{OA} >$ 2,300 pc/h

Exhibit 13-12
Estimating Speed at Off-Ramp (Diverge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS - (FFS - 42)D_S$	
	$D_S = 0.883 + 0.00009v_R - 0.013S_{FR}$	
Outer lanes of freeway	$S_O = 1.097\,FFS$	$v_{OA} < 1{,}000$ pc/h
	$S_O = 1.097\,FFS - 0.0039\,(v_{OA} - 1{,}000)$	$v_{OA} \geq 1{,}000$ pc/h

Exhibit 13-13
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)	$S = \dfrac{v_{R12} + v_{OA}N_O}{\left(\dfrac{v_{R12}}{S_R}\right) + \left(\dfrac{V_{OA}N_O}{S_O}\right)}$
Average speed for off-ramp (diverge) junctions (mi/h)	$S = \dfrac{v_{12} + v_{OA}N_O}{\left(\dfrac{v_{12}}{S_R}\right) + \left(\dfrac{v_{OA}N_O}{S_O}\right)}$

While many (but not all) of the variables in Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 have been defined previously, all are defined 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_{R12} = 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);

v_F = demand flow rate on freeway immediately upstream of the ramp influence area (pc/h);

N_O = number of outer lanes on the freeway (1 for a six-lane freeway; 2 for an eight-lane freeway);

M_S = speed index for on-ramps (merge areas); this is simply an intermediate computation that simplifies the equations; and

D_S = speed index for off-ramps (diverge areas); this is simply an intermediate computation that simplifies the equations.

Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 only apply to stable flow conditions. Consult Chapter 10 for analysis of oversaturated conditions.

The equations in Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 apply only to cases in which operation is stable (LOS A–E). Analysis of operational details for cases in which LOS F is present relies on deterministic queuing approaches, as presented in Chapter 10, Freeway Facilities.

Flow rates in outer lanes can be higher than the value cited for basic freeway segments. The basic freeway segment values represent averages across all freeway lanes, not flow rates in a single lane or a subset of lanes. The methodology herein allows flows in outer lanes to be as high as 2,700 pc/h/ln. The equations for average speed in outer lanes were based on a database that included average outer lane flows as high as 2,988 pc/h/ln while still maintaining stable flow. Values over 2,700 pc/h/ln, however, are unusual and cannot be expected in the majority of situations.

In addition, the equations of Exhibit 13-11 do not allow a predicted speed over the FFS for merge areas. For diverge areas at low flow rates, however, the average speed in outer lanes may marginally exceed the FFS. As with average lane flow rates, the FFS is stated as an average across all lanes, and speeds in individual lanes can exceed this value. Despite this, the average speed of all vehicles S should be limited to a maximum value equal to the FFS.

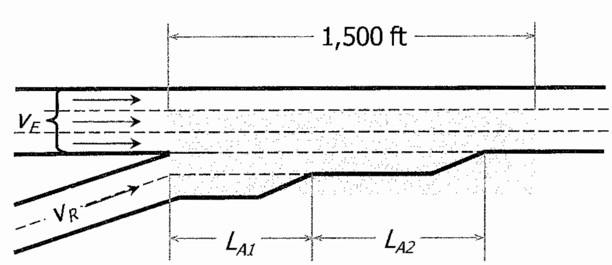
SPECIAL CASES

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 general methodology is modified to account for special situations. These modifications are discussed in the sections that follow.

Two-Lane On-Ramps

Exhibit 13-14 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 13-14
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.

Two-lane on-ramp–freeway junctions, however, 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, this value is replaced by the effective length of both acceleration lanes L_{Aeff} from Equation 13-23:

$$L_{Aeff} = 2L_{A1} + L_{A2}$$

Equation 13-23

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 13-14.

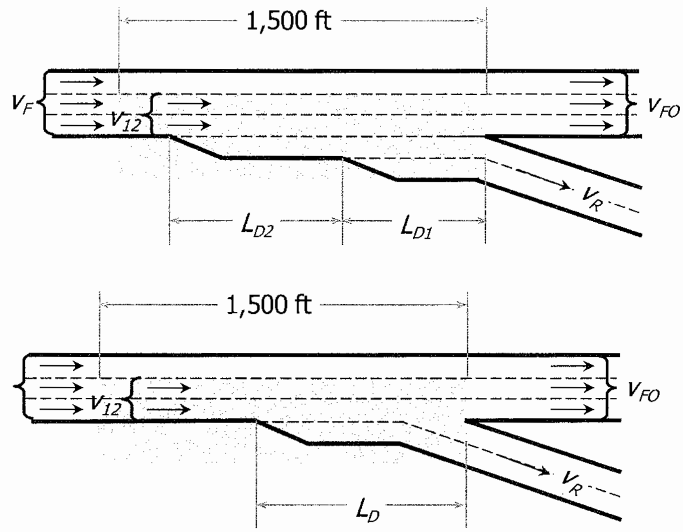
Two-Lane Off-Ramps

Two common types of diverge geometries are in use with two-lane off-ramps, as shown in Exhibit 13-15. 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 13-15
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 13-24:

Equation 13-24

$$L_{Deff} = 2L_{D1} + L_{D2}$$

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 13-15.

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; 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 (3).

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 13-16.

Exhibit 13-16
Adjustment Factors for Left-Hand Ramp–Freeway Junctions

Freeway Size	Adjustment Factor for Left-Hand Ramps: On-Ramps	Adjustment Factor for Left-Hand 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 (4). Exhibit 13-17 shows the recommended values for flow rate in Lane 5 of these segments.

Exhibit 13-17
Expected Flow in Lane 5 of a 10-Lane Freeway Immediately Upstream of a Ramp–Freeway Junction

On-Ramps: Approaching Freeway Flow v_F (pc/h)	On-Ramps: Approaching Lane 5 Flow v_5 (pc/h)	Off-Ramps: Approaching Freeway Flow v_F (pc/h)	Off-Ramps: 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 13-25:

Equation 13-25

$$v_{F4eff} = v_F - v_5$$

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).

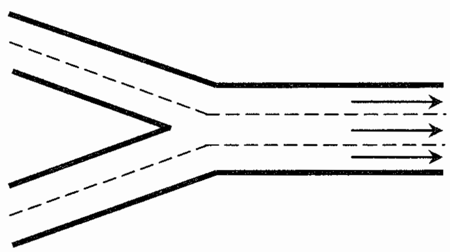
There is no calibrated procedure for adapting the methodology of this chapter to freeways with more than five lanes in one direction. The approach of Equation 13-25 is, however, conceptually adaptable to such situations. A local calibration of the amount of traffic using Lanes 5+ would be needed. The remaining flow could then be modeled as if it were taking place on a four-lane (one direction) segment.

Major Merge Areas

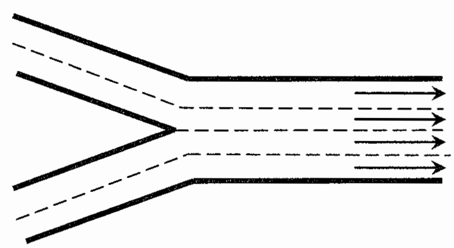
A major merge area is one in which two primary roadways, each having multiple lanes, merge to form a single freeway segment. Such junctions occur when two freeways join to form a single freeway or when a major multilane high-speed ramp joins with a freeway. Major merges are different from one- and two-lane 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 13-18.

Exhibit 13-18
Major Merge Areas Illustrated



(a) Major Merge with One Lane Dropped



(b) Major Merge with No Lane Dropped

LOS cannot be determined for major merge areas.

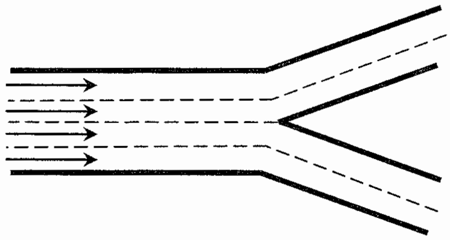
There are no effective models of performance for a major merge area. Therefore, analysis is limited to checking capacities on the approaching legs and the downstream freeway segment. A merge failure would be indicated by a v/c ratio in excess of 1.00. LOS cannot be determined for major merge areas. Problems in major merge areas usually result from insufficient capacity of the downstream freeway segment.

Major Diverge Areas

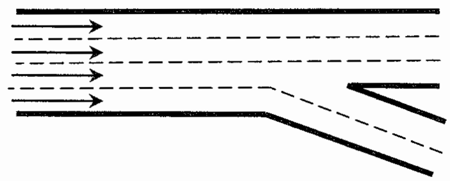
The two common geometries for major diverge areas are illustrated in Exhibit 13-19. In the first case, the number of lanes leaving the diverge area is the same as the number entering it. In the second, the number of lanes leaving the diverge area is one more than the number entering it.

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 13-19
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 13-26:

Equation 13-26

$$D_{MD} = 0.0175\left(\frac{v_F}{N}\right)$$

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.

The result can be compared with the criteria of Exhibit 13-2 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).

Effect of Ramp Control at On-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 (5) 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.

OVERLAPPING RAMP INFLUENCE AREAS

Whenever a series of ramps on a freeway is analyzed, the 1,500-ft ramp influence areas could overlap. In such cases, the operation in the overlapping region is determined by the ramp influence area having the highest density.

3. 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, however, are possible.

DEFAULT VALUES

A comprehensive presentation of potential default values for uninterrupted-flow facilities is provided elsewhere (*6*). Chapter 10, Freeway Facilities, provides a summary of the default values for freeways. These defaults cover the key characteristics of peak hour factor (PHF) and percent heavy vehicles (%HV) on freeways. Recommendations are based on geographical region, population, and time of day. All general freeway default values may be applied to the analysis of ramp–freeway junctions in the absence of field data or projections of conditions.

Because of the number of variables involved in the analysis of ramps, which have been discussed previously, it is difficult to base an analysis on too many default values. Clearly, all demand flow rates must be specified, even if they are projections.

Ramp geometric characteristics cover a variety of conditions; default values should be avoided if possible.

Similarly, geometric characteristics of ramps cover a wide variety of conditions. If absolutely necessary, the following additional default values may be applied to a ramp junction analysis:

- Length of acceleration lane L_A = 800 ft,
- Length of deceleration lane L_D = 400 ft,
- FFS of ramp S_{FR} = 35 mi/h, and
- Driver population factor f_p = 1.00.

Obviously, as the number of default values used in any analysis increases, the accuracy of the result becomes more approximate, and the result may be significantly different from the actual outcome (depending on local conditions). If locally calibrated default values are available, they may be substituted for the values above.

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, it is important to establish the segment of freeway over which ramp junctions are to be analyzed. 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.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design analysis.

Operational Analysis

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 attempting to optimize the others.

It is possible to program a spreadsheet to complete such an analysis, providing scenario results 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 for freeway facilities in Chapter 10 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 13-27

$$SV_i = SF_i \times PHF$$

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 13-2 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 a number of ways in which service flow rates and volumes can be considered:

- 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.

Any of these are viable concepts for establishing a ramp service flow rate or service volume.

A number of factors influence the service volume or flow rate result; each situation must be individually considered.

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, %HV, length of acceleration or deceleration lane(s), ramp FFS, and any relevant data for adjacent ramps. It is, therefore, virtually impossible to define a representative "typical" case with broadly applicable results. Each case must be individually considered.

The Example Problems section includes an example of how ramp junction service flow rates and service 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}.

Strengths of the HCM Procedure

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 a body of expert consensus. Most simulation packages will not include the level of detail present in this methodology concerning the ramp itself and its adjacent upstream and downstream ramps.

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 and demands on the upstream and downstream ramps.
- It produces a single deterministic estimate of density, which is important for some purposes, such as development impact review.

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

A list of the HCM's limitations for freeway merge and diverge segments is provided in Exhibit 13-20.

Exhibit 13-20
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 Chapter 10 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.

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, it is not clear how a simulator should address the partial presence of vehicles in the link to ensure compatibility with the HCM. 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, it is important to consider the following:

- 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, as 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, as increased driver variability in the simulator will generally increase the average density.

With regard to capacity, 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. In some cases it may be necessary to provide virtual detectors at specific points on the simulated segment 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 11 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 required 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 this 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, while the second evaluates the impacts of converting the leftmost lane of the mainline into an HOV lane.

4. EXAMPLE PROBLEMS

Exhibit 13-21
List of Example Problems

Example Problem	Title	Type of Analysis
1	Isolated One-Lane, Right-Hand On-Ramp to a Four-Lane Freeway	Operational analysis
2	Two Adjacent Single-Lane, Right-Hand Off-Ramps on a Six-Lane Freeway	Operational analysis
3	One-Lane On-Ramp Followed by a One-Lane Off-Ramp on an Eight-Lane Freeway	Operational analysis
4	Single-Lane, Left-Hand On-Ramp on a Six-Lane Freeway	Special case
5	Service Flow Rates and Service Volumes for an Isolated On-Ramp on a Six-Lane Freeway	Service flow rates and service volumes

EXAMPLE PROBLEM 1: ISOLATED ONE-LANE, RIGHT-HAND ON-RAMP TO A FOUR-LANE FREEWAY

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Isolated location (no adjacent ramps to consider)
- One-lane ramp roadway and junction
- Four-lane freeway (two lanes in each direction)
- Upstream freeway demand volume = 2,500 veh/h
- Ramp demand volume = 550 veh/h
- 10% trucks, 0% RVs on the freeway
- 5% trucks, 0% RVs on the ramp
- Acceleration lane = 740 ft
- FFS, freeway = 60 mi/h
- FFS, ramp = 45 mi/h
- Level terrain for freeway and ramp
- Peak hour factor = 0.90
- Drivers are regular commuters

Comments

All input parameters are known, so no default values are needed or used. Adjustment factors for heavy vehicles and driver population are found in Chapter 11, Basic Freeway Segments.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

Demand volumes are given for the freeway and the ramp. The PHF is specified. The driver population factor for commuters is 1.00 (Chapter 11), while the heavy vehicle adjustment factor is computed as follows:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Truck and RV presence is given. The value of E_T for level terrain is 1.5 (Chapter 11). On the basis of these values, the freeway and ramp demand volumes are converted as follows:

For the freeway:

$$f_{HV} = \frac{1}{1 + 0.10\,(1.5 - 1)} = 0.952$$

$$v_F = \frac{2{,}500}{0.90 \times 0.952 \times 1.00} = 2{,}918 \text{ pc/h}$$

For the ramp:

$$f_{HV} = \frac{1}{1 + 0.05\,(1.5 - 1)} = 0.976$$

$$v_F = \frac{550}{0.90 \times 0.976 \times 1.00} = 626 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Ramp Influence Area with Equation 13-2 and Exhibit 13-6

$$v_{12} = v_F \times P_{FM}$$

The freeway flow rate was computed in Step 1. The value of P_{FM} is found in Exhibit 13-6. For a four-lane freeway, the value is 1.00. Then

$$v_{12} = 2{,}918 \times 1.00 = 2{,}918 \text{ pc/h}$$

Because there are no outer lanes on a four-lane freeway, there is no need to check this result for reasonableness.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

The critical capacity checkpoint for a single-lane on-ramp is the downstream freeway segment:

$$v_{FO} = v_F + v_R = 2{,}918 + 626 = 3{,}544 \text{ pc/h}$$

The capacity of a four-lane freeway (two lanes in one direction) with an FFS of 60 mi/h is given in Exhibit 13-8. The capacity is 4,600 pc/h, which is more than the demand flow of 3,544 pc/h. The capacity of a one-lane ramp with an FFS of 45 mi/h is given in Exhibit 13-10 as 2,100 pc/h, which is well in excess of the ramp demand flow of 626 pc/h. The maximum desirable flow rate entering the ramp influence area is also 4,600 pc/h, again more than 3,544. Thus, the operation of the segment is expected to be stable. LOS F does not exist.

Step 4: Compute Density and Find LOS by Using Equation 13-21 and Exhibit 13-2

The estimated density in the ramp–freeway junction is estimated by using Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

$$D_R = 5.475 + (0.00734 \times 626) + (0.0078 \times 2{,}918) - (0.00627 \times 740) = 28.2 \text{ pc/mi/ln}$$

From Exhibit 13-2, this is LOS D, but the result is close to the LOS C boundary.

Step 5: Compute Merge Area Speed as Supplemental Information by Using Exhibit 13-11

Since there are no outer lanes present on a four-lane freeway, only the speed within the ramp influence area should be computed:

$$S_R = FFS - (FFS - 42) M_S$$

$$M_S = 0.321 + 0.0039 e^{(v_{R12}/1{,}000)} - 0.002 \left(L_A S_{FR} / 1{,}000 \right)$$

$$M_S = 0.321 + 0.0039 e^{(3{,}544/1{,}000)} - 0.002 (740 \times 45 / 1{,}000) = 0.389$$

$$S_R = 60 - (60 - 42) \times 0.389 = 53.0 \text{ mi/h}$$

Discussion

The results indicate that the merge area operates in a stable fashion, with some deterioration in density and speed due to merging operations.

EXAMPLE PROBLEM 2: TWO ADJACENT SINGLE-LANE, RIGHT-HAND OFF-RAMPS ON A SIX-LANE FREEWAY

The Facts

The following information concerning demand volumes and geometries is available for this problem:

- Two consecutive one-lane, right-hand off-ramps
- Six-lane freeway with FFS = 60 mi/h
- Rolling terrain for freeway and both ramps
- 5% trucks on freeway and both ramps; 0% RVs
- First ramp FFS = 40 mi/h
- Second ramp FFS = 25 mi/h
- Drivers are regular commuters
- Freeway demand volume = 4,500 veh/h (immediately upstream of the first off-ramp)
- First ramp demand volume = 300 veh/h
- Second ramp demand volume = 500 veh/h
- Distance between ramps = 750 ft
- First ramp deceleration lane length = 500 ft

- Second ramp deceleration lane length = 300 ft
- Peak hour factor = 0.95

Comments

The solution will use adjustment factors for heavy vehicle presence and driver population selected from Chapter 11, Basic Freeway Segments. All input parameters are specified, so no default values are needed or used.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

In this case, three demand volumes must be converted: the freeway volume immediately upstream of the first ramp and the two ramp demand volumes. Since all demands include 5% trucks and no RVs, only a single heavy vehicle adjustment factor will be needed. From Chapter 11, the appropriate value of E_T for rolling terrain is 2.5. For drivers who are regular commuters, the appropriate value of f_p is 1.00.

Then

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1)} = 0.930$$

and

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

$$v_F = \frac{4{,}500}{0.95 \times 0.930 \times 1.00} = 5{,}093 \text{ pc/h}$$

$$v_{R1} = \frac{300}{0.95 \times 0.930 \times 1.00} = 340 \text{ pc/h}$$

$$v_{R2} = \frac{500}{0.95 \times 0.930 \times 1.00} = 566 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Two Ramp Influence Areas by Using Equation 13-13 and Exhibit 13-7

Because there are two consecutive off-ramps under consideration, the first will have to consider the impact of the second on its operations, and the second will have to consider the impact of the first.

First Off-Ramp

From Exhibit 13-7, flow in Lanes 1 and 2 of the freeway is estimated by using Equation 13-11 or Equation 13-9, depending on whether the impact of the downstream off-ramp is significant. This is determined by computing the equivalence distance by using Equation 13-13:

$$L_{EQ} = \frac{v_D}{1.15 + 0.000032 v_F - 0.000369 v_{R1}}$$

$$L_{EQ} = \frac{566}{1.15 + (0.000032 \times 5{,}093) - (0.000369 \times 340)} = 657 \text{ ft}$$

Since the actual distance between ramps, 750 ft, is greater than the equivalence distance of 657 ft, the ramp may be treated as if it were isolated, with Equation 13-9:

$$P_{FD} = 0.760 - 0.000025 v_F - 0.000046 v_{R1}$$

$$P_{FD} = 0.760 - (0.000025 \times 5093) - (0.000046 \times 340) = 0.617$$

Then

$$v_{12} = v_R + (v_F - v_R) P_{FD}$$

$$v_{12} = 340 + (5{,}093 - 340) \times 0.617 = 3{,}273 \text{ pc/h}$$

Because a six-lane freeway includes one outer lane (Lane 3), the reasonableness of the predicted lane distribution of arriving freeway vehicles should be checked. The flow rate in Lane 3 is 5,093 – 3,273 = 1,820 pc/h. The average flow per lane in Lanes 1 and 2 is 3,273/2 = 1,637 pc/h (rounded to the nearest pc). Then:

Is $v_3 > 2{,}700$ pc/h/ln?	**No**
Is $v_3 > 1.5 \times (1{,}637) = 2{,}456$ pc/h/ln?	**No**

Since both checks for reasonable lane distribution are passed, the computed value of v_{12} for the first off-ramp is accepted as 3,273 pc/h.

Second Off-Ramp

From Exhibit 13-7, the second off-ramp should be analyzed by using Equation 13-9, which is for an isolated off-ramp. Adjacent upstream off-ramps do not affect the lane distribution of arriving vehicles at a downstream off-ramp.

The freeway flow approaching Ramp 2, however, includes the freeway flow approaching Ramp 1, less the flow rate of vehicles exiting the freeway at Ramp 1. Therefore, the freeway flow rate approaching Ramp 2 is as follows:

$$v_{F2} = 5{,}093 - 340 = 4{,}753 \text{ pc/h}$$

Then

$$P_{FD} = 0.760 - (0.000025 \times 4753) - (0.000046 \times 566) = 0.615$$

$$v_{12} = 566 + (4{,}753 - 566) \times 0.615 = 3{,}141 \text{ pc/h}$$

Again, because there is an outer lane on a six-lane freeway, the reasonableness of this estimate must be checked. The flow rate in the outer lane v_3 is 4,753 – 3,141 = 1,612 pc/h. The average flow rate in Lanes 1 and 2 is 3,141/2 = 1,571 pc/h (rounded). Then:

Is $v_3 > 2{,}700$ pc/h/ln? **No**

Is $v_3 > 1.5 \times 1{,}571 = 2{,}357$ pc/h/ln? **No**

Once again, the predicted lane distribution of arriving vehicles is reasonable, and v_{12} is taken to be 3,141 pc/h.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

Because two off-ramps are involved in this segment, there are several capacity checkpoints:

- Total freeway flow upstream of the first off-ramp (the point at which maximum freeway flow exists),
- Capacity of both off-ramps, and
- Maximum desirable flow rates entering each of the two off-ramp influence areas.

These comparisons are shown in Exhibit 13-22. Note that freeway capacity is based on a freeway with FFS = 60 mi/h. The first ramp capacity is based on a ramp FFS of 40 mi/h and the second on a ramp FFS of 25 mi/h.

Exhibit 13-22
Capacity Checks for Example Problem 2

Item	Capacity (pc/h) Exhibit 13-8, Exhibit 13-10	Demand Flow Rate (pc/h)	Problem?
Freeway flow rate	6,900	5,093	No
First off-ramp	2,000	340	No
Second off-ramp	1,900	566	No
Max. v_{12} first ramp	4,400	3,373	No
Max. v_{12} second ramp	4,400	3,141	No

None of the capacity values are exceeded, so operation of these ramp junctions will be stable, and LOS F does not occur.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-22 and Exhibit 13-2

Because there are two off-ramps, two ramp influence areas are involved, and two ramp influence area densities will be computed.

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

$$D_{R1} = 4.252 + (0.0086 \times 3{,}273) - (0.009 \times 500) = 27.9 \text{ pc/mi/ln}$$

$$D_{R2} = 4.252 + (0.0086 \times 3{,}141) - (0.009 \times 300) = 28.6 \text{ pc/mi/ln}$$

From Exhibit 13-2, both of these ramp influence areas operate very close to the boundary between LOS C and LOS D (28.0 pc/mi/ln). Ramp 1 operates in LOS C, while Ramp 2 operates in LOS D.

While it makes virtually no difference in this case, note that the two ramp influence areas overlap. The influence area of the first off-ramp extends 1,500 ft upstream. The influence area of the second off-ramp also extends 1,500 ft upstream. Given that the ramps are only 750 ft apart, the second ramp influence area overlaps the first for 750 ft (immediately upstream of the first diverge point). Normally, the worst of the two levels of service would be applied to this 750-ft overlap. In this case, the levels of service are the same. Indeed, the predicted densities are virtually equal, so the impact of the overlap is minimal, and the predicted values are not really affected.

Step 5: Compute Diverge Area Speeds as Supplemental Information by Using Exhibit 13-12 and Exhibit 13-13

Because these ramps are on a six-lane freeway with an outer lane, it is possible to estimate the speed within each ramp influence area, the speed in the outer lane adjacent to each ramp influence area, and the weighted average of the two.

First Off-Ramp

The speed within the first ramp influence area is computed as follows:

$$D_S = 0.883 + 0.00009 v_R - 0.013 S_{FR}$$

$$D_S = 0.883 + (0.00009 \times 3{,}273) - (0.013 \times 40) = 0.394$$

$$S_R = FFS - (FFS - 42) D_S = 60 - (60 - 42) \times 0.394 = 52.9 \text{ mi/h}$$

The flow rate in the outer lane (v_{OA}) is 5,093 – 3,273 = 1,820 pc/h/ln. The average speed in this outer lane is computed as follows:

$$S_O = 1.097 FFS - 0.0039\,(v_{OA} - 1{,}000)$$

$$S_O = (1.097 \times 60) - 0.0039 \times (1{,}820 - 1{,}000) = 62.6 \text{ mi/h}$$

The average speed in Lane 3 is predicted to be slightly higher than the FFS of the freeway. This is not uncommon, since through vehicles at higher speeds use Lane 3 to avoid congestion in the ramp influence area. The average speed across all lanes, however, should not be higher than the FFS. In this case, the average speed across all lanes is computed as follows:

$$S = \frac{3{,}273 + (1{,}820 \times 1)}{\left(\dfrac{3{,}273}{52.9}\right) + \left(\dfrac{1{,}820 \times 1}{62.0}\right)} = 56.0 \text{ mi/h}$$

This result is, as expected, less than the FFS of the freeway.

Second Off-Ramp

The speed in the second ramp influence area is computed as follows:

$$D_S = 0.883 + (0.00009 \times 566) - (0.013 \times 25) = 0.609$$

$$S_R = 60 - (60 - 42) \times 0.609 = 49.0 \text{ mi/h}$$

Lane 3 has a demand flow rate of 4,753 – 3,141 = 1,612 pc/h/ln. The average speed in this outer lane is computed as follows:

$$S_O = (1.097 \times 60) - 0.0039 \times (1{,}612 - 1{,}000) = 63.4 \text{ mi/h}$$

The average speed across all freeway lanes is

$$S = \frac{3{,}141 + (1{,}612 \times 1)}{\left(\frac{3{,}141}{49.0}\right) + \left(\frac{1{,}612 \times 1}{63.4}\right)} = 53.1 \text{ mi/h}$$

Discussion

The speed results in this case are interesting. While densities are similar for both ramps, the density is somewhat higher and the speed somewhat lower in the second influence area. This is primarily the result of a shorter deceleration lane and a lower ramp FFS (25 mi/h versus 40 mi/h). In both cases, the average speed in the outer lane is higher than the FFS, which applies as an average across all lanes.

Since the operation is stable, there is no special concern here, short of a significant increase in demand flows. LOS is technically D but falls just over the LOS C boundary. This is a case in which the step-function LOS assigned may imply an operation poorer than actually exists. It emphasizes the importance of knowing not only the LOS but also the value of the service measure that produces it.

EXAMPLE PROBLEM 3: ONE-LANE ON-RAMP FOLLOWED BY A ONE-LANE OFF-RAMP ON AN EIGHT-LANE FREEWAY

The Facts

The following information is available concerning this pair of ramps to be analyzed:

- Eight-lane freeway with an FFS of 65 mi/h
- One-lane, right-hand on-ramp with an FFS of 30 mi/h
- One-lane, right-hand off-ramp with an FFS of 25 mi/h
- Distance between ramps = 1,300 ft
- Acceleration lane on Ramp 1 = 260 ft
- Deceleration lane on Ramp 2 = 260 ft
- Level terrain on freeway and both ramps
- 10% trucks, no RVs on freeway and off-ramp
- 5% trucks, no RVs on on-ramp
- Freeway flow rate (upstream of first ramp) = 5,500 veh/h
- On-ramp flow rate = 400 veh/h
- Off-ramp flow rate = 600 veh/h
- PHF = 0.90
- Drivers are regular commuters

Comments

As with previous example problems, the conversion of demand volumes to flow rates requires adjustment factors selected from Chapter 11, Basic Freeway

Segments. All pertinent information is given, and no default values will be applied.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

Three demand volumes must be converted to flow rates under equivalent ideal conditions: the freeway volume immediately upstream of the first ramp junction, the first ramp volume, and the second ramp volume. Because the freeway segment under study has level terrain, the value of E_T will be 1.5 for all volumes. Because the drivers are regular commuters, the driver population factor, f_p, is 1.00.

Then, for the freeway demand volume:

$$f_{HV} = \frac{1}{1+0.10(1.5-1)} = 0.952$$

$$v_F = \frac{5{,}500}{0.90 \times 0.952 \times 1.00} = 6{,}419 \text{ pc/h}$$

For the on-ramp demand volume:

$$f_{HV} = \frac{1}{1+0.05(1.5-1)} = 0.976$$

$$v_{R1} = \frac{400}{0.90 \times 0.976 \times 1.00} = 455 \text{ pc/h}$$

For the off-ramp demand volume:

$$f_{HV} = \frac{1}{1+0.10(1.5-1)} = 0.952$$

$$v_{R2} = \frac{600}{0.90 \times 0.952 \times 1.00} = 700 \text{ pc/h}$$

In the remaining computations, these converted demand flow rates are used as input values.

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Two Ramp Influence Areas by Using Equation 13-2 and Exhibit 13-6 for the On-Ramp and Equation 13-8 and Exhibit 13-7 for the Off-Ramp

Once again, the situation involves a pair of adjacent ramps. Each ramp must consider the potential impact of the other on its operations. Because the ramps are on an eight-lane freeway (four lanes in each direction), Exhibit 13-6 and Exhibit 13-7 indicate that each ramp is considered as if it were isolated.

First Ramp (On-Ramp)

Equation 13-2 and Exhibit 13-6 apply to on-ramps. Exhibit 13-6 presents two possible equations for use in estimating v_{12} on the basis of the value of v_F/S_{FR}. In this case, the value is 6,419/30 = 210.6 > 72. Therefore, Equation 13-5 is used, giving the following:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.2178 - 0.000125 v_R$$

$$P_{FM} = 0.2178 - (0.000125 \times 455) = 0.161$$

$$v_{12} = 6{,}419 \times 0.161 = 1{,}033 \text{ pc/h}$$

Because the eight-lane freeway includes two outer lanes in each direction, the reasonableness of this prediction must be checked. The average flow per lane in Lanes 1 and 2 is 1,033/2 = 517 pc/h/ln (rounded). The flow in the two outer lanes, Lanes 3 and 4, is 6,419 – 1,033 = 5,386 pc/h. The average flow per lane in Lanes 3 and 4 is, therefore, 5,386/2 = 2,693 pc/h/ln. Then:

Is v_{av34} > 2,700 pc/h/ln? **No**

Is v_{av34} > 1.5 × 517 = 776 pc/h/ln? **Yes**

The predicted lane distribution, therefore, is not reasonable. Too many vehicles are placed in the two outer lanes compared with Lanes 1 and 2. Equation 13-19 is used to produce a more reasonable distribution:

$$v_{12a} = \left(\frac{v_F}{2.50}\right) = \left(\frac{6{,}419}{2.50}\right) = 2{,}568 \text{ pc/h}$$

On the basis of this adjusted value, the number of vehicles now assigned to the two outer lanes is 6,419 – 2,568 = 3,851 pc/h.

Second Ramp (Off-Ramp)

Equation 13-8 and Exhibit 13-7 apply to off-ramps. Exhibit 13-7 shows that the value of P_{FD} for off-ramps on eight-lane freeways is a constant: 0.436. As the methodology is based on regression analysis of a database, the recommendation of a constant reflects a small sample size in that database. Note also that the freeway flow approaching the second ramp is the sum of the freeway flow approaching the first ramp and the on-ramp flow that is now also on the freeway, or 6,419 + 455 = 6,874 pc/h. The flow rate in Lanes 1 and 2 is now easily computed by using Equation 13-8:

$$v_{12} = v_R + (v_F - v_R) P_{FD}$$

$$v_{12} = 700 + (6{,}874 - 700) \times 0.436 = 3{,}392 \text{ pc/h}$$

Because there are two outer lanes on this eight-lane freeway, the reasonableness of this estimate must be checked. The average flow per lane in Lanes 1 and 2 is 3,392/2 = 1,696 pc/h/ln. The total flow in Lanes 3 and 4 of the freeway is 6,874 – 3,392 = 3,482 pc/h, or an average flow rate per lane of 3,482/2 = 1,741 pc/h/ln.

Is v_{av34} > 2,700 pc/h/ln? **No**

Is $v_{av34} > 1.5 \times 1{,}696 = 2{,}544$ pc/h/ln? **No**

Therefore, the estimated value of v_{12} is deemed reasonable and is carried forward in the computations.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

Because there are two ramps in this segment, there are five capacity checkpoints to consider:

- The freeway flow rate at its maximum point—which in this case is between the on- and off-ramp, since this is the only location where both on- and off-ramp vehicles are on the freeway.
- The capacity of the on-ramp.
- The capacity of the off-ramp.
- The maximum desirable flow entering the on-ramp influence area.
- The maximum desirable flow entering the off-ramp influence area.

These comparisons are shown in Exhibit 13-23. The capacity of the freeway is based on an eight-lane freeway with an FFS of 65 mi/h. The capacity of the on-ramp is based on an FFS of 30 mi/h, and the capacity of the off-ramp is based on an FFS of 25 mi/h.

Exhibit 13-23
Capacity Checks for Example Problem 3

Item	Capacity (pc/h) Exhibit 13-8, Exhibit 13-10	Demand Flow Rate (pc/h)	Problem?
Freeway flow rate	9,400	6,874	No
First on-ramp	1,900	345	No
Second off-ramp	1,900	700	No
Max. v_{R12} first ramp	4,600	2,568 + 455 = 3,023	No
Max. v_{12} second ramp	4,400	3,392	No

There are no capacity concerns, since all demands are well below the associated capacities or maximum desirable values. LOS F is not present in any part of this segment, and operations are expected to be stable.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-21, Equation 13-22, and Exhibit 13-2

Equation 13-21 is used to find the density in the first on-ramp influence area:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

$$D_R = 5.475 + (0.00734 \times 455) + (0.0078 \times 2{,}568) - (0.00627 \times 260) = 27.2 \text{ pc/mi/ln}$$

Equation 13-22 is used to find the density in the second off-ramp influence area:

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

$$D_R = 4.252 + (0.0086 \times 3{,}391) - (0.009 \times 260) = 31.1 \text{ pc/mi/ln}$$

From Exhibit 13-2, both of these ramp influence areas operate very close to the boundary between LOS C and LOS D (28 pc/mi/ln). Ramp 1 operates in LOS C, while Ramp 2 operates in LOS D.

Because the on-ramp influence area extends 1,500 ft downstream, the off-ramp influence area extends 1,500 ft upstream, and the two ramps are only 1,300 ft apart, the distance between the ramps is included in both. Therefore, the more pessimistic prediction of LOS D for the off-ramp governs the operation. Oddly, the additional 200 ft of the off-ramp influence area is actually upstream of the on-ramp, and the additional 200 ft of the on-ramp influence area is downstream of the off-ramp.

Step 5: Compute Merge and Diverge Area Speeds as Supplemental Information by Using Exhibit 13-11 and Exhibit 13-12

Because of the eight-lane freeway, speeds should be estimated for the two ramp influence areas, for the outer lanes (Lanes 3 and 4) adjacent to the ramp influence area, and for all vehicles—the weighted average of the other two speeds.

First Ramp (On-Ramp)

Equations for estimation of average speed in an on-ramp influence area and in outer lanes adjacent to it are taken from Exhibit 13-11.

$$M_S = 0.321 + 0.0039e^{(v_{R12}/1,000)} - 0.002(L_A S_{FR}/1,000)$$

$$M_S = 0.321 + 0.0039e^{(3,032/1,000)} - 0.002(260/30) = 0.385$$

$$S_R = FFS - (FFS - 42)M_S = 65 - (65 - 42) \times 0.385 = 56.2 \text{ mi/h}$$

Since the average outer lane demand flow rate is 3,851/2 = 1,926 pc/h/ln, which is greater than 500 pc/h/ln and less than 2,300 pc/h/ln, the outer speed is estimated as follows:

$$S_O = FFS - 0.0036\,(v_{OA} - 500)$$

$$S_O = 65 - 0.0036(1,926 - 500) = 59.9 \text{ mi/h}$$

The weighted average speed of all vehicles is

$$S = \frac{3,032 + (1,926 \times 2)}{\left(\frac{3,032}{56.2}\right) + \left(\frac{1,926 \times 2}{59.9}\right)} = 58.2 \text{ mi/h}$$

Second Ramp (Off-Ramp)

For off-ramps, equations for estimation of average speed are drawn from Exhibit 13-12. At the second ramp, the flow in Lanes 1 and 2 has been computed as 3,392 pc/h or 1,696 pc/h/ln, while the flow in Lanes 3 and 4 is 3,482 pc/h, or 1,741 pc/h/ln. Then

$$D_S = 0.883 + 0.00009v_R - 0.013S_{FR}$$

$$D_S = 0.883 + (0.00009 \times 700) - (0.013 \times 25) = 0.621$$

$$S_R = FFS - (FFS - 42)D_S$$

$$S_R = 65 - (65 - 42) \times 0.621 = 50.7 \text{ mi/h}$$

Because the average flow in the outer lanes is greater than 1,000 pc/h/ln, the average speed of vehicles in the outer lanes (Lanes 3 and 4) is as follows:

$$S_O = 1.097FFS - 0.0039\left(v_{OA} - 1{,}000\right)$$

$$S_O = (1.097 \times 65) - 0.0039\,(1{,}741 - 1{,}000) = 68.4 \text{ mi/h}$$

The weighted average speed of all vehicles is

$$S = \frac{3{,}392 + (1{,}741 \times 2)}{\left(\dfrac{3{,}392}{50.7}\right) + \left(\dfrac{1{,}741 \times 2}{68.4}\right)} = 58.3 \text{ mi/h}$$

Discussion

As noted previously, between the ramps, the influence areas of both ramps fully overlap. Since a higher density is predicted for the off-ramp influence area, and LOS D results, this density should be applied to the entire area between the two ramps.

The speed results are also interesting. The slower speeds within the off-ramp influence area will also control the overlap area. On the other hand, the speed results indicate a higher average speed for all vehicles associated with the off-ramp than the speed associated with the on-ramp. This is primarily due to the much larger disparity between speeds within the ramp influence area and in outer lanes when the off-ramp is considered. The speed differential is more than 20 mi/h for the off-ramp, as opposed to a little more than 3 mi/h for the on-ramp. This is not entirely unexpected. At diverge junctions, vehicles in outer lanes tend to face less turbulence than those in outer lanes near merge junctions. All off-ramp vehicles must be in Lanes 1 and 2 for some distance before exiting the freeway. On-ramp vehicles, on the other hand, can execute as many lane changes as they wish—consistent with safety and sanity—and more of them may wind up in outer lanes within 1,500 ft of the junction point.

Thus, the total operation of this two-ramp segment is expected to be LOS D, with speeds of approximately 50 mi/h in Lanes 1 and 2 and approximately 70 mi/h in Lanes 3 and 4.

EXAMPLE PROBLEM 4: SINGLE-LANE, LEFT-HAND ON-RAMP ON A SIX-LANE FREEWAY

The Facts

- One-lane, left-side on-ramp on a six-lane freeway (three lanes in each direction)
- Freeway demand volume upstream of ramp = 4,000 veh/h
- On-ramp demand volume = 500 veh/h
- 15% trucks, no RVs on freeway
- 5% trucks, no RVs on ramp
- Freeway FFS = 65 mi/h
- Ramp FFS = 30 mi/h

- Acceleration lane = 820 ft
- Level terrain on freeway and ramp
- Drivers are regular commuters

Comments

This is a special application of the ramp analysis methodology presented in this chapter. For left-hand ramps, the flow rate in Lanes 1 and 2 (v_{12}) is initially computed as if it were a right-hand ramp. Exhibit 13-16 is then used to convert this result to an estimate of the flow in Lanes 2 and 3 (v_{23}), since these are the two leftmost lanes that will be involved in the merge. In effect, the ramp influence area is, in this case, Lanes 3 and 4 and the acceleration lane for a distance of 1,500 ft downstream of the merge point.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

From Chapter 11, Basic Freeway Segments, the passenger car equivalent E_T for trucks in level terrain is 1.5. The driver population adjustment factor f_p for regular commuters is 1.00.

For the freeway demand volume:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV} = \frac{1}{1 + 0.15(1.5 - 1)} = 0.930$$

$$v_F = \frac{4{,}000}{0.90 \times 0.93 \times 1.00} = 4{,}779 \text{ pc/h}$$

For the ramp demand volume:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1)} = 0.976$$

$$v_R = \frac{500}{0.90 \times 0.976 \times 1.00} = 569 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 2 and 3 Immediately Upstream of the Ramp Influence Area by Using Equation 13-2 and Exhibit 13-6

To estimate flow in the two left lanes, the flow normally expected in Lanes 1 and 2 for a similar right-hand ramp must first be computed. From Exhibit 13-6, for an isolated on-ramp on a six-lane freeway, Equation 13-4 is used:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.5775 + 0.000028 L_A$$

$$P_{FM} = 0.5775 + (0.000028 \times 820) = 0.600$$

$$v_{12} = 4{,}779 \times 0.600 = 2{,}867 \text{ pc/h}$$

From Exhibit 13-16, the adjustment factor applied to this result to find the estimated flow rate in Lanes 2 and 3 is 1.12. Therefore:

$$v_{23} = 2{,}867 \times 1.12 = 3{,}211 \text{ pc/h}$$

While, strictly speaking, the reasonableness criteria for lane distribution do not apply to left-hand ramps, they can be applied very approximately. In this case, the single "outer lane" (which is now Lane 1) would have a flow rate of 4,779 – 3,211 = 1,568 pc/h. This is not greater than 2,700 pc/h/ln, nor is it greater than 1.5 times the average flow in Lanes 2 and 3 (1.5 × 3,211/2 = 2,408 pc/h/ln). Thus, even if the reasonableness criteria were approximately applied in this case, no violation would exist.

The remaining computations proceed for the left-hand ramp, with the substitution of v_{34} for v_{12} in all algorithms used.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

For this case, there are three simple checkpoints:

- The principal capacity checkpoint is the total demand flow rate downstream of the merge, 4,779 + 569 = 5,348 pc/h. From Exhibit 13-8, for a six-lane freeway with an FFS of 65 mi/h, the capacity is 7,050 pc/h, well over the demand flow rate.
- The ramp roadway capacity should also be checked by using Exhibit 13-10. For a single-lane ramp with an FFS of 30 mi/h, the capacity is 1,900 pc/h, which is much greater than the demand flow rate of 569 pc/h.
- Finally, the maximum flow entering the ramp influence area should be checked. In this case, a left-hand ramp, the total flow entering the ramp influence area is the freeway flow remaining in Lanes 2 and 3 plus the ramp flow rate. Thus, the total flow entering the ramp influence area is 3,211 + 569 = 3,780 pc/h, which is lower than the maximum desirable flow rate of 4,600 pc/h, shown in Exhibit 13-8.

Thus, there are no capacity problems at this merge point, and stable operations are expected. LOS F will not result from the stated conditions.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-21 and Exhibit 13-2

The density in the ramp influence area is found by using Equation 13-21, except v_{23} replaces v_{12} because of the left-hand ramp placement:

$$D_S = 5.475 + 0.00734 v_R + 0.0078 v_{23} - 0.00627 L_A$$

$$D_S = 5.475 + (0.00734 \times 569) + (0.0078 \times 3{,}211) - (0.00627 \times 820) = 29.6 \text{ pc/mi/ln}$$

From Exhibit 13-2, this is LOS D.

Step 5: Compute Merge and Diverge Area Speeds as Supplemental Information by Using Exhibit 13-11 and Exhibit 13-13

The speed estimation algorithms were calibrated for right-hand ramps, and the estimation algorithms for "outer lane(s)" assume that these are the leftmost lanes. Thus, for a left-hand ramp, these computations must be considered approximate at best.

By using the equations in Exhibit 13-11 and Exhibit 13-13, the following results are obtained:

$$M_S = 0.321 + 0.0039e^{(3{,}780/1{,}000)} - 0.002(820 \times 30/1{,}000) = 0.443$$

$$S_R = 65 - (65 - 42) \times 0.443 = 54.8 \text{ mi/h}$$

$$S_O = 65 - 0.0036\ (1{,}568 - 500) = 61.2 \text{ mi/h}$$

$$S = \frac{3{,}780 + (1{,}568 \times 1)}{\left(\frac{3{,}780}{54.8}\right) + \left(\frac{1{,}568 \times 1}{61.2}\right)} = 56.5 \text{ mi/h}$$

While traffic in the outer lane is predicted to travel somewhat faster than traffic in the lanes in the ramp influence area (which includes the acceleration lane), the approximate nature of the speed result for left-hand ramps makes it difficult to draw any firm conclusions concerning speed behavior.

Discussion

This example problem is typical of the way the situations in the Special Cases section are treated. Modifications as specified are applied to the standard algorithms used for single-lane, right-hand ramp junctions. In this case, operations are acceptable, but in LOS D—though not far from the LOS C boundary. Because the left-hand lanes are expected to carry freeway traffic flowing faster than right-hand lanes, right-hand ramps are normally preferable to left-hand ramps when they can be provided without great difficulty.

EXAMPLE PROBLEM 5: SERVICE FLOW RATES AND SERVICE VOLUMES FOR AN ISOLATED ON-RAMP ON A SIX-LANE FREEWAY

The Facts

The following facts have been established for this situation:

- Single-lane, right-hand on-ramp with an FFS of 40 mi/h
- Six-lane freeway (three lanes in each direction) with an FFS of 70 mi/h
- Level terrain for freeway and ramp
- 12% trucks, 3% RVs on freeway
- 5% trucks, 2% RVs on ramp
- Peak hour factor = 0.87
- Drivers are regular users of the facility
- Acceleration lane = 1,000 ft

Comments

This example illustrates the computation of service flow rates and service volumes for a ramp–freeway junction. The case selected is relatively straightforward to avoid cluttering the illustration with extraneous complications that have been addressed in other example problems.

Two approaches will be demonstrated:

1. The ramp demand flow rate will be stated as a fixed percentage of the arriving freeway flow rate. The service flow rates and service volumes are expressed as arriving freeway flow rates that result in the threshold densities within the ramp influence area that define the limits of the various levels of service. For this computation, the ramp flow is set at 10% of the approaching freeway flow rate.
2. A fixed freeway demand flow rate will be stated, with service flow rates and service volumes expressed as ramp demand flow rates that result in the threshold densities within the ramp influence area that define the limits of the various levels of service. For this computation, the approaching freeway flow rate is set at 4,000 veh/h.

For LOS E, density does not define the limiting value of service flow rate, which is analogous to capacity for ramp–freeway junctions. It is defined as the flow that results in capacity being reached on the downstream freeway segment or ramp roadway.

Since all algorithms in this methodology are calibrated for passenger cars per hour under equivalent ideal conditions, initial computations are made in those terms. Results are then converted to service flow rates by using the appropriate heavy vehicle and driver population adjustment factors. Service flow rates are then converted to service volumes by multiplying by the peak hour factor.

From Exhibit 13-2, the following densities define the limits of LOS A–D:

LOS A	10 pc/mi/ln
LOS B	20 pc/mi/ln
LOS C	28 pc/mi/ln
LOS D	35 pc/mi/ln

From Exhibit 13-8 and Exhibit 13-10, capacity (or the threshold for LOS E) occurs when the downstream freeway flow rate reaches 7,200 pc/h (FFS = 70 mi/h) or when the ramp flow rate reaches 2,000 pc/h (ramp FFS = 40 mi/h).

Case 1: Ramp Demand Flow Rate = 0.10 Freeway Demand Flow Rate

Equation 13-21 defines the density in an on-ramp influence area as follows:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

In this case

v_R = 0.10 v_F

L_A = 1,000 ft

Equation 13-2 and Exhibit 13-6 give the following:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.5775 + 0.000028 L_A$$

$$P_{FM} = 0.5775 + (0.000028 \times 1{,}000) = 0.6055$$

$$v_{12} = 0.6055 v_F$$

Substitution of these values into Equation 13-21 gives

$$D_R = 5.475 + (0.00734 \times 0.10 v_F) + (0.0078 \times 0.6055 v_F) - (0.00627 \times 1{,}000)$$

$$D_R = 5.475 + 0.000734 v_F + 0.00472 v_F - 6.27$$

$$D_R = 0.005454 v_F - 0.795$$

$$v_F = \frac{D_R + 0.795}{0.005454}$$

This equation can now be solved for threshold values of v_F for LOS A through D by using the appropriate threshold values of density. The results will be in terms of service flow rates under equivalent ideal conditions:

$$v_F(\text{LOS A}) = \frac{10 + 0.795}{0.005454} = 1{,}979 \text{ pc/h}$$

$$v_F(\text{LOS B}) = \frac{20 + 0.795}{0.005454} = 3{,}813 \text{ pc/h}$$

$$v_F(\text{LOS C}) = \frac{28 + 0.795}{0.005454} = 5{,}280 \text{ pc/h}$$

$$v_F(\text{LOS D}) = \frac{35 + 0.795}{0.005454} = 6{,}563 \text{ pc/h}$$

At capacity, the limiting flow rate occurs when the downstream freeway segment is 7,200 pc/h. If the ramp flow rate is 0.10 of the approaching freeway flow rate, then

$$v_{FO} = 7{,}200 = v_F + 0.10 v_F = 1.10 v_F$$

$$v_F(\text{LOS E}) = \frac{7{,}200}{1.10} = 6{,}545 \text{ pc/h}$$

This must be checked to ensure that the ramp flow rate ($0.10 \times 6{,}545 = 655$ pc/h) does not exceed the ramp capacity of 2,000 pc/h. Since it does not, the computation stands.

Note, however, that the LOS E (capacity) threshold is lower than the LOS D threshold. This indicates that LOS D operation cannot be achieved at this location. Before densities reach the 35 pc/h/ln threshold for LOS D, the capacity of the merge junction has been reached. Thus, there is no service flow rate or service volume for LOS D.

The computed values, as noted, are in terms of passenger cars per hour under equivalent ideal conditions. To convert these to service flow rates in vehicles per hour under prevailing conditions, they must be multiplied by the heavy vehicle adjustment factor and the driver population factor. The approaching freeway flow includes 12% trucks and 3% RVs. For level terrain (Chapter 11, Basic Freeway Segments), $E_T = 1.5$ and $E_R = 1.2$. Then

$$f_{HV} = \frac{1}{1 + 0.12(1.5 - 1) + 0.03(1.2 - 1)} = 0.938$$

The driver population factor f_p for regular facility users is 1.00. Service volumes are obtained by multiplying service flow rates by the specified PHF, 0.87. These computations are illustrated in Exhibit 13-24.

Exhibit 13-24
Illustrative Service Flow Rates and Service Volumes Based on Approaching Freeway Demand

LOS	Service Flow Rate, Ideal Conditions (pc/h)	Service Flow Rate, Prevailing Conditions (SF) (veh/h)	Service Volume (SV) (veh/h)
A	1,979	1,979 × 0.938 × 1 = 1,856	1,856 × 0.87 = 1,615
B	3,813	3,813 × 0.938 × 1 = 3,577	3,577 × 0.87 = 3,112
C	5,280	5,280 × 0.938 × 1 = 4,953	4,953 × 0.87 = 4,309
D	NA	NA	NA
E	6,545	6,545 × 0.938 × 1 = 6,139	6,139 × 0.87 = 5,341

The service flow rates and service volumes shown in Exhibit 13-24 are stated in terms of the approaching freeway demand.

Case 2: Approaching Freeway Demand Volume = 4,000 veh/h

In this case, the approaching freeway demand will be held constant, and service flow rates and service volumes will be stated in terms of the ramp demand that can be accommodated at each LOS.

Since the freeway demand is stated in terms of an hourly volume in mixed vehicles per hour, it will be converted to passenger cars per hour under equivalent ideal conditions for use in the algorithms of this methodology:

$$v_F = \frac{V_F}{PHF \times f_{HV} \times f_p} = \frac{4{,}000}{0.87 \times 0.938 \times 1} = 4{,}902 \text{ pc/h}$$

The density is estimated by using Equation 13-20, and the variable P_{FM}—which is not dependent on v_R—remains 0.6055 as in Case 1. With a fixed value of freeway demand:

$$v_{12} = 0.6055 \times 4{,}902 = 2{,}968 \text{ pc/h}$$

Then, by using Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + (0.0078 \times 2{,}968) - (0.00627 \times 1{,}000)$$

$$D_R = 22.355 + 0.00734 v_R$$

$$v_R = \frac{D_R - 22.355}{0.00734}$$

It is apparent from this equation that neither LOS A ($D_R = 10$ pc/mi/ln) nor LOS B ($D_R = 20$ pc/mi/ln) can be achieved with a fixed freeway demand flow of 4,902 pc/h.

For LOS C and D:

$$v_R(\text{LOS C}) = \frac{28 - 22.355}{0.00734} = 769 \text{ pc/h}$$

$$v_R(\text{LOS D}) = \frac{35 - 22.355}{0.00734} = 1{,}723 \text{ pc/h}$$

Capacity, the limit of LOS E, occurs when the downstream freeway flow reaches 7,200 pc/h. With a fixed freeway demand:

$$v_{FO} = 7{,}200 = 4{,}902 + v_R$$

$$v_R(\text{LOS E}) = 7{,}200 - 4{,}902 = 2{,}298 \text{ pc/h}$$

This, however, violates the capacity of the ramp roadway, which is 2,000 pc/h. Thus, the limiting ramp flow rate for LOS E is set at 2,000 pc/h.

As in Case 1, these values are all stated in terms of passenger cars per hour under equivalent ideal conditions. They are converted to service flow rates by multiplying by the appropriate heavy vehicle and driver population adjustment factors. Since the ramp has a composition different from that of the approaching freeway flow, its adjustment must be recomputed:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1) + 0.02(1.2 - 1)} = 0.972$$

Service flow rates are converted to service volumes by multiplying by the peak hour factor. These computations are illustrated in Exhibit 13-25.

Exhibit 13-25
Illustrative Service Flow Rates and Service Volumes Based on a Fixed Freeway Demand

LOS	Service Flow Rate, Ideal Conditions (pc/h)	Service Flow Rate, Prevailing Conditions (SF) (veh/h)	Service Volume (SV) (veh/h)
A	NA	NA	NA
B	NA	NA	NA
C	769	769 × 0.972 × 1 = 747	747 × 0.87 = 650
D	1,723	1,723 × 0.972 × 1 = 1,675	1,675 × 0.87 = 1,457
E	2,000	2,000 × 0.972 × 1 = 1,944	1,944 × 0.87 = 1,691

These service flow rates and service volumes are based on a constant upstream arriving freeway demand and are stated in terms of limiting on-ramp demands for that condition.

Discussion

As this illustration shows, many considerations are involved in estimating service flow rates and service volumes for ramp–freeway junctions, not the least of which is specifying how such values should be defined. The concept of service flow rates and service volumes at specific ramp–freeway junctions is of limited utility. Since many of the details that affect the estimates will not be determined until final designs are prepared, operational analysis of the proposed design may be more appropriate.

Case 2 could have applications in considering how to time ramp meters. Appropriate limiting ramp flows can be estimated by using the same approach as for service volumes and service flow rates.

5. REFERENCES

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

1. Roess, R. P., and J. M. Ulerio. *Capacity of Ramp–Freeway Junctions.* Final Report, NCHRP Project 3-37. Polytechnic University, Brooklyn, N.Y., Nov. 1993.
2. Roess, R. P., E. S. Prassas, and W. R. McShane. *Traffic Engineering,* 3rd ed. Pearson Prentice Hall, Upper Saddle River, N.J., 2004.
3. Leisch, J. E. *Capacity Analysis Techniques for Design and Operation of Freeway Facilities.* Federal Highway Administration, Washington, D.C., 1974.
4. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets.* Washington, D.C., 2004.
5. Elefteriadou, L. *A Probabilistic Model of Breakdown at Freeway–Merge Junctions.* Doctoral dissertation. Polytechnic University, Brooklyn, N.Y., June 1994.
6. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses.* Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 14
MULTILANE HIGHWAYS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 14, Multilane Highways, addresses capacity and level-of-service (LOS) analysis for uninterrupted-flow segments of surface multilane highways. In general, uninterrupted flow may exist on a multilane highway if there are 2 mi or more between traffic signals. Where signals are more closely spaced, the facility should be analyzed as an urban street.

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 signalized intersections are analyzed by using the methodology of this chapter. Isolated signalized intersections should be analyzed with the methodology of Chapter 18, Signalized Intersections.

LOS procedures are provided for both automobiles and bicycles. The automobile methodology is based on the results of NCHRP Project 3-33 (*1*), and bicycle LOS is based on research conducted for the Florida Department of Transportation (2). The same methodology for bicycle LOS is used for both multilane and two-lane highways; readers interested in details of the bicycle methodology should refer to Chapter 15, Two-Lane Highways.

TYPES OF MULTILANE HIGHWAYS

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 are used on some multilane highways. These highways may be divided by one of various median types, may be undivided (with only a centerline separating the directions of flow), or may have a two-way left-turn lane (TWLTL). They are typically located in suburban areas, leading into central cities, or along high-volume rural corridors, connecting two cities or two activity centers that generate a substantial number of daily trips. Exhibit 14-1 illustrates common types of multilane highways.

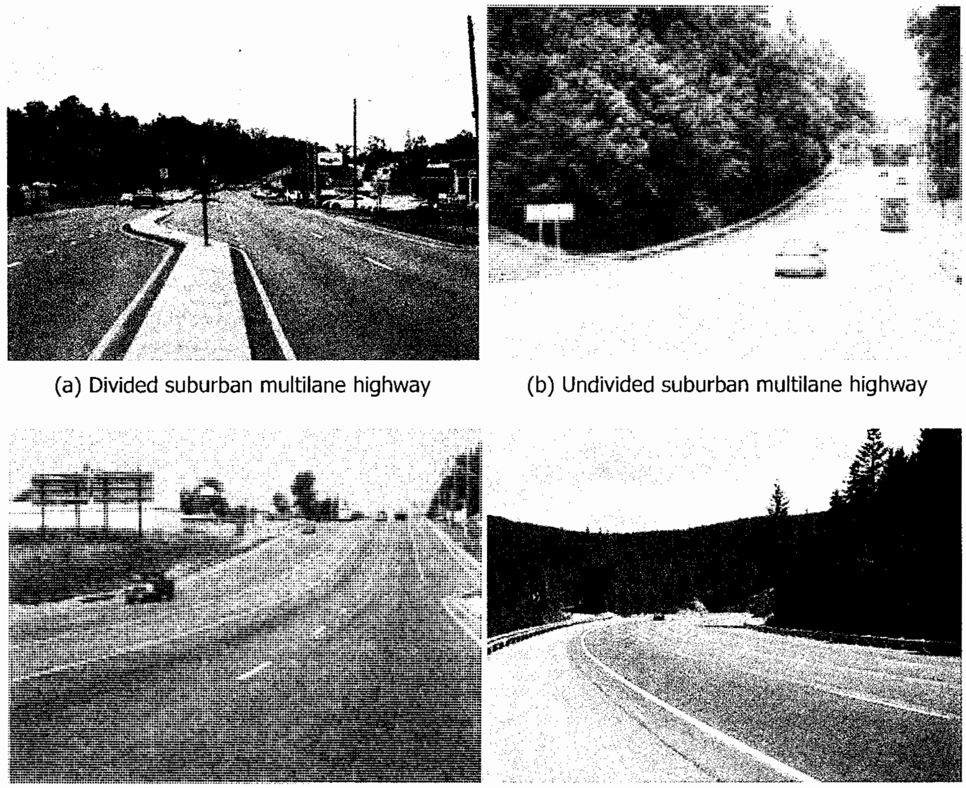
Traffic volumes on multilane highways vary widely but often have demand in the range of 15,000 to 40,000 veh/day. In some cases, volumes as high as 100,000 veh/day have been observed when access across the median is restricted and when major crossings are grade-separated. Bicycles are typically permitted on multilane highways, and multilane highways often serve as primary routes for both commuter cyclists (on suburban highways) and recreational cyclists (on rural highways).

BASE CONDITIONS

Base conditions include good weather, good visibility, and no incidents or accidents. These conditions are always assumed to exist.

The base conditions under which the full capacity of a multilane highway segment is achieved include good weather, good visibility, no incidents or accidents, no work zone activity, and no pavement defects that would affect operations. This chapter's methodology assumes that these conditions exist. If any of these conditions do not exist, the speed, LOS, and capacity of the multilane highway segment can be expected to be worse than the predictions by this methodology.

Exhibit 14-1
Multilane Highways



(a) Divided suburban multilane highway

(b) Undivided suburban multilane highway

(c) Suburban multilane highway with TWLTL

(d) Undivided rural multilane highway

Base conditions include the following conditions; the methodology can be adjusted to address situations in which these conditions do not exist:

Base conditions include 0% heavy vehicles and a driver population composed of regular users of the highway.

The methodology provides adjustments for situations in which these conditions do not apply.

- No heavy vehicles, such as trucks, buses, and recreational vehicles (RVs), in the traffic stream; and
- A driver population composed primarily of regular users who are familiar with the facility.

Characteristics such as lane width, total lateral clearance (TLC), median type, and access-point density will have an impact on the free-flow speed (FFS) of the facility. Curves describing operations under base conditions, however, account for differing FFSs.

FLOW CHARACTERISTICS UNDER BASE CONDITIONS

More severe geometric characteristics and the existence of access points are two key differences that result in lower multilane highway speeds and capacities than those of freeways with similar cross sections.

Uninterrupted flow on multilane highways is in most ways similar to that on basic freeway segments (Chapter 11). Several factors are different, however. 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 also tends to be more severe than that of basic freeway segments because of the lower speed expectations. Last, 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.

Exhibit 14-2 shows speed–flow characteristics of multilane highway segments for various FFSs. Equations describing these curves are shown in Exhibit 14-3.

Curves are shown for FFSs between 45 mi/h and 60 mi/h. Because FFSs can vary widely, it is recommended that the FFS of a multilane highway segment be estimated to the nearest 5 mi/h, as follows:

The FFS of a multilane highway segment should be rounded to the nearest 5 mi/h.

42.5 mi/h ≤ FFS < 47.5 mi/h: use FFS = 45 mi/h,

47.5 mi/h ≤ FFS < 52.5 mi/h: use FFS = 50 mi/h,

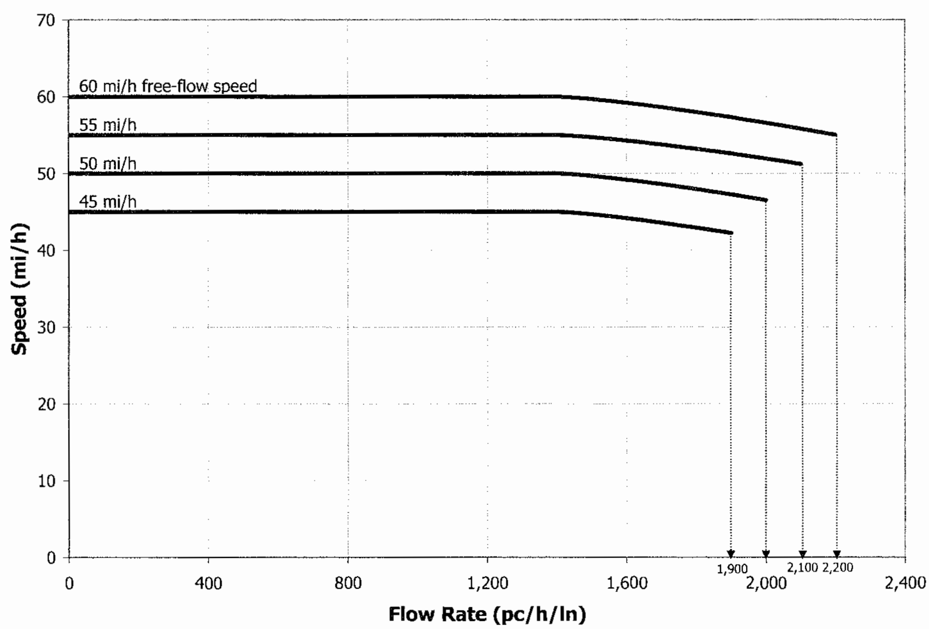
52.5 mi/h ≤ FFS < 57.5 mi/h: use FFS = 55 mi/h,

57.5 mi/h ≤ FFS < 62.5 mi/h: use FFS = 60 mi/h.

For multilane highway segments, speeds remain constant until they reach 1,400 pc/h/ln, after which speeds decline with further increases in flow rate.

Flow rates over 1,400 pc/h/ln result in speeds below the highway's FFS.

Exhibit 14-2
Speed–Flow Curves for Multilane Highways Under Base Conditions



Note: Maximum densities for LOS E occur at a *v/c* ratio of 1.00. These are 40, 41, 43, and 45 pc/mi/ln for FFSs of 60, 55, 50, and 45 mi/h, respectively.

Exhibit 14-3
Equations Describing Speed–Flow Curves in Exhibit 14-2

FFS (mi/h)	For $v_p \leq$ 1,400 pc/h/ln, S(mi/h)	For $v_p >$ 1,400 pc/h/ln, S(mi/h)
60	60	$60-\left[5.00\times\left(\frac{v_p-1400}{800}\right)^{1.31}\right]$
55	55	$55-\left[3.78\times\left(\frac{v_p-1400}{700}\right)^{1.31}\right]$
50	50	$50-\left[3.49\times\left(\frac{v_p-1400}{600}\right)^{1.31}\right]$
45	45	$45-\left[2.78\times\left(\frac{v_p-1400}{500}\right)^{1.31}\right]$

CAPACITY OF MULTILANE HIGHWAY SEGMENTS

Multilane highways with higher FFSs will also have higher base capacities. As most highways do not operate under base conditions, observed capacities will usually be lower than the base capacity.

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

The capacity of a multilane highway segment under base conditions varies with the FFS. For 60-mi/h FFS, the capacity is 2,200 pc/h/ln. For lesser FFSs, capacity diminishes. For 55-mi/h FFS, the capacity is 2,100 pc/h/ln; for 50-mi/h FFS, 2,000 pc/h/ln; and for 45-mi/h FFS, 1,900 pc/h/ln.

These values represent national norms. Capacity varies stochastically, and any given location could have a larger or smaller value. In addition, capacity refers to the average flow rate across all lanes. Thus, a two-lane (in one direction) multilane highway segment with a 60-mi/h FFS would have an expected capacity of $2 \times 2{,}200 = 4{,}400$ pc/h. This flow would not be uniformly distributed in the two lanes. Thus, one lane could have stable flows in excess of 2,200 pc/h/ln.

LOS FOR MULTILANE HIGHWAY SEGMENTS

Automobile Mode

Automobile LOS is defined by density.

Automobile LOS for multilane highway segments are defined in Exhibit 14-4. Because speeds are constant through a broad range of flow rates, LOS are defined on the basis of density, which is a measure of the proximity of vehicles to each other in the traffic stream.

Exhibit 14-4
Automobile LOS for Multilane Highway Segments

LOS	FFS (mi/h)	Density (pc/mi/ln)
A	All	>0–11
B	All	>11–18
C	All	>18–26
D	All	>26–35
E	60	>35–40
	55	>35–41
	50	>35–43
	45	>35–45
F	Demand Exceeds Capacity	
	60	>40
	55	>41
	50	>43
	45	>45

LOS thresholds for multilane highways are the same as those on freeways for LOS A–D. However, multilane highway capacity (the LOS E–F boundary) occurs at lower densities.

For LOS A through D, the criteria are the same as those for basic freeway segments. This classification is appropriate, since both represent multilane uninterrupted flow. The boundary between LOS E and F, however, represents capacity. For multilane highways, capacity occurs at varying densities, depending on the FFS. The density at capacity ranges from 40 pc/mi/ln for 60-mi/h FFS to 45 pc/mi/ln for 45-mi/h FFS.

LOS F is determined when the demand flow rate exceeds capacity. When this occurs, the methodology does not produce a density estimate. Thus, although density in such cases will be above the thresholds shown, specific values cannot be determined.

Exhibit 14-5 shows LOS thresholds in relation to the base speed–flow curves.

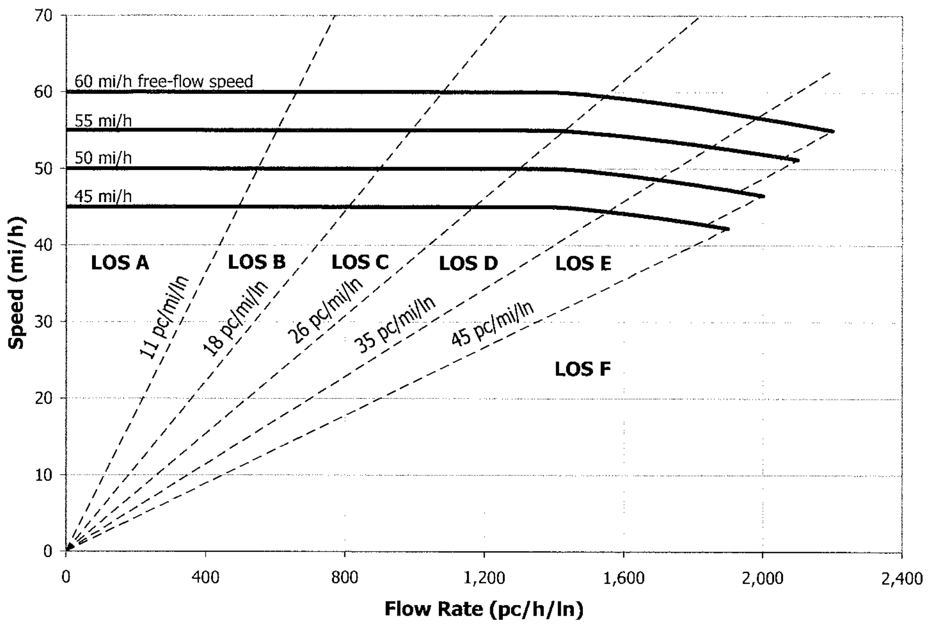


Exhibit 14-5
LOS on Base Speed–Flow Curves

Automobile LOS Described

The descriptions of LOS for basic freeway segments given in Chapter 11 are also generally applicable to multilane highways. Vehicles entering the highway from a direct access point are an additional factor on multilane highways; these vehicles are not present on basic freeway segments and could result in a breakdown of flow at high flow rates.

The LOS thresholds for multilane highways reflect the collective professional judgment of the members of the Transportation Research Board's Highway Capacity and Quality of Service Committee. The upper values shown for LOS F (40 to 45 pc/mi/ln, depending on the FFS) represent the maximum density at which sustained flows at capacity are expected to occur. Breakdown (LOS F) conditions on multilane highways occur whenever the highway's demand exceeds its capacity.

Bicycle Mode

Bicycle LOS for multilane highway segments are based on a bicycle LOS score, which is in turn based on a traveler-perception index. Chapter 15, Two-Lane Highways, provides details about this index, which is identical for two-lane highways and multilane highways. The LOS ranges for bicycles on multilane highways are given in Exhibit 14-6.

Bicycle LOS is based on a traveler-perception index score. Details are given in Chapter 15.

Exhibit 14-6
Bicycle LOS on 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

Automobile Mode

Analysis of a multilane highway segment requires details concerning the geometric characteristics of the segment and the demand characteristics of the users of the segment. This section presents the required input data for the basic freeway segment methodology; specifics about individual parameters are given in Section 2, Methodology.

Data Describing Multilane Highway Segment

The following information concerning the geometric features of the multilane highway segment is needed to conduct an analysis:

- FFS: 45 to 60 mi/h;
- Number of lanes (one direction): two or three;
- Lane width: 10 ft to more than 12 ft;
- Right-side lateral clearance: 0 ft to more than 6 ft;
- Median- (left-) side lateral clearance: 0 ft to more than 6 ft;
- Access-point density: 0 to 40 points/mi;
- Terrain: level, rolling, or mountainous; or length and percent grade of specific grades; and
- Type of median: divided, TWLTL, or undivided.

Data Describing Demand

The following information is required concerning the users of the multilane highway segment:

- Demand during the analysis hour; or daily demand, *K*-factor, and *D*-factor;
- Heavy-vehicle presence (percent trucks and buses, percent RVs): 0%–100% in general terrain or 0%–25% for specific grades;
- Peak hour factor (PHF): up to 1.00; and
- Driver-population factor: 0.85–1.00.

Length of Analysis Period

The period for any multilane highway analysis is generally the critical 15-min period within the peak hour. The methodology can be applied to any 15-min period, however.

If demand volumes are used, demand flow rates are estimated through the use of the PHF. When 15-min volumes are directly measured, the worst analysis period within the hour 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.

Bicycle Mode

The following data are required to evaluate bicycle LOS on a multilane highway; the ranges of values used in the development of the bicycle LOS model (2) are also shown:

- 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: 45 to 50 mi/h,
- Heavy-vehicle percentage: 0% to 2%, and
- Pavement condition: present serviceability rating of 1 to 5.

2. METHODOLOGY

This methodology is used to analyze the capacity, LOS, lane requirements, and impacts of traffic and design features on uninterrupted-flow segments of rural and suburban multilane highways.

LIMITATIONS OF METHODOLOGY

Automobile Mode

The methodology of this chapter does not take into account the following conditions:

- The negative impacts of poor weather conditions, traffic accidents or incidents, railroad crossings, or construction operations;
- Interference caused by parking on the shoulders of the multilane highway;
- 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;
- Differences between various types of median barriers and the difference between the impacts of a median barrier and a TWLTL;
- FFS below 45 mi/h or higher than 60 mi/h;
- Significant presence of on-street 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 facilities with more than 2 mi between traffic signals. When the 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.

Uninterrupted-flow 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.

Bicycle Mode

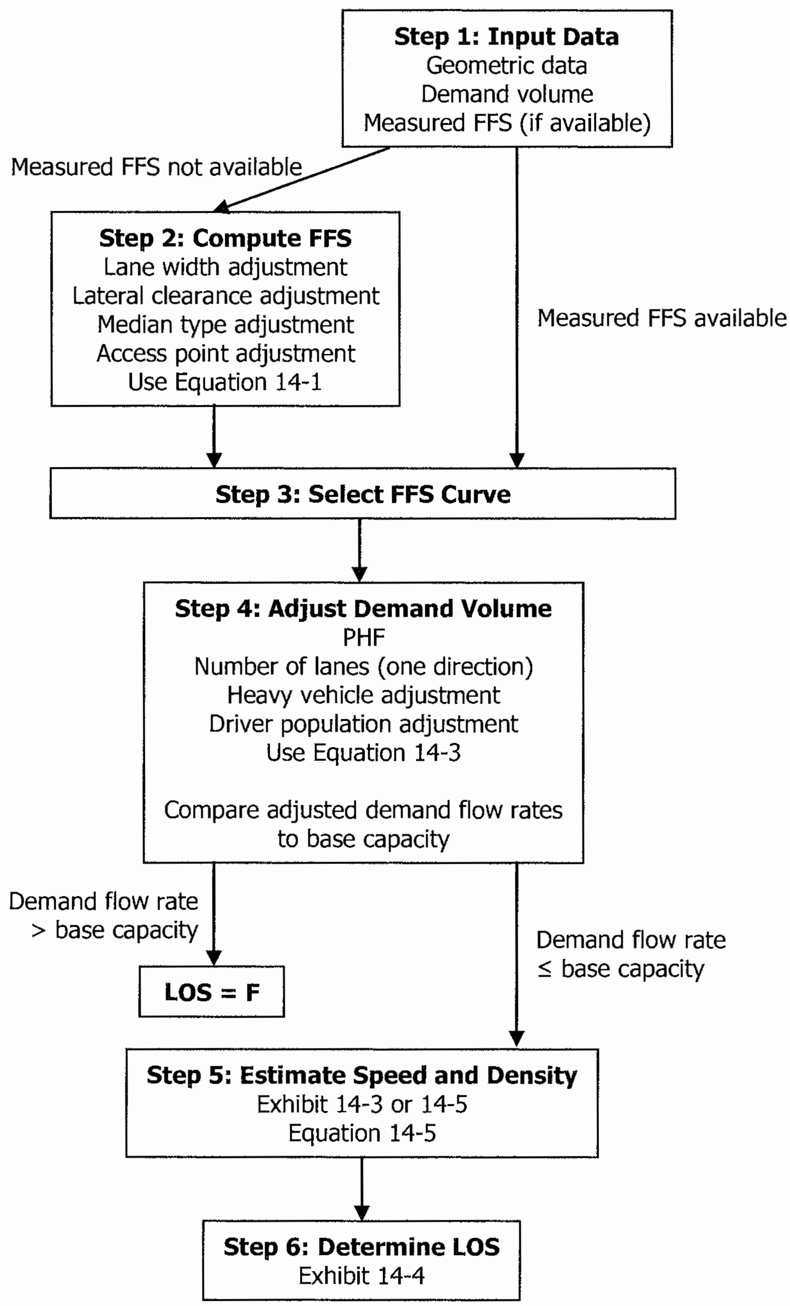
Although the bicycle LOS model has been successfully applied to rural multilane highways, users should be aware that conditions on many of those highways are outside the range of values used to develop the model.

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.

AUTOMOBILE MODE

Exhibit 14-7 provides an overview of this chapter's computational methodology for the automobile mode. It shows a typical operational analysis in which the LOS is determined for a specified set of geometric and traffic conditions. The methodology can also be used, as described in this chapter's Applications section, to determine the number of lanes needed to provide a target LOS, as well as to determine service flow rates, service volumes, and daily service volumes.

Exhibit 14-7
Overview of Multilane Highway Methodology for Automobile Mode



Step 1: Input Data

For a typical operational analysis, the analyst must specify (with either site-specific or default values) demand volume; number and width of lanes; right-side and median lateral clearance; type of median; roadside access points per mile; percent of heavy vehicles, such as trucks, buses, and RVs; PHF; terrain; and driver population factor.

Step 2: Compute FFS

FFS can be determined directly from field measurements or can be estimated as described below.

Field Measurement

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 1,400 pc/h/ln). For a specific multilane highway segment, speeds are virtually constant in this range of flow rates. If the FFS can be field measured, that determination is preferable. If the FFS is measured directly, no adjustments are applied to the measured value.

The speed study should be conducted at a location representative of the segment at a time when flow rates are less than 1,400 pc/h/ln. The speed study should measure the speeds of all passenger cars or use a systematic sample (e.g., every tenth 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 a standard traffic engineering publication (*3*).

Estimation

It is not possible to make field measurements for future facilities, and field measurement may not be possible or practical for all existing ones. In such cases, the segment's FFS may be estimated by using Equation 14-1, which is based on the physical characteristics of the segment under study:

Equation 14-1

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

where

$BFFS$ = base FFS for multilane highway segment (mi/h);

FFS = FFS of basic freeway segment (mi/h);

f_{LW} = adjustment for lane width, from Exhibit 14-8 (mi/h);

f_{LC} = adjustment for TLC, from Exhibit 14-9 (mi/h);

f_M = adjustment for median type, from Exhibit 14-10 (mi/h); and

f_A = adjustment for access-point density, from Exhibit 14-11 (mi/h).

Base FFS

This methodology covers multilane highway segments with FFS ranging from 45 mi/h to 60 mi/h. The most significant value in Equation 14-1 is the 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 horizontal and vertical alignment of the highway, not on the impacts of lane widths, lateral clearances, median type, and access points. The design speed may be used as the BFFS if it is available.

Although speed limits are not always uniformly set, the BFFS 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

Average lane widths less than 12 ft reduce the FFS.

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 narrow average lane widths are shown in Exhibit 14-8.

Exhibit 14-8
Adjustment to FFS for Average Lane Width

Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Adjustment for Lateral Clearance

Clearance restrictions on either the right or left side of the highway reduce the FFS.

The adjustment for lateral clearance 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 because there is a separate adjustment for the type of median that accounts for the impact of an undivided highway on FFS.

Equation 14-2

Equation 14-2 is used to determine TLC:

$$TLC = LC_R + LC_L$$

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 14-9 shows the reduction in FFS due to lateral obstructions on the multilane highway.

Exhibit 14-9 Adjustment to FFS for Lateral Clearances

Four-Lane Highways		Six-Lane Highways	
TLC (ft)	**Reduction in FFS (mi/h)**	**TLC (ft)**	**Reduction in FFS (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

The FFS is reduced on undivided highways.

The adjustment for type of median is given in Exhibit 14-10. Undivided multilane highways reduce the BFFS by 1.6 mi/h.

Exhibit 14-10 Adjustment to FFS for Median Type

Median Type	Reduction in FFS, f_M (mi/h)
Undivided	1.6
TWLTL	0.0
Divided	0.0

Adjustment for Access-Point Density

FFS is reduced as the access-point density increases.

Exhibit 14-11 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 with little activity, should not be used to determine access-point density.

Exhibit 14-11 Adjustment to FFS for Access-Point Density

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, it might be appropriate to include intersection approaches and driveways on both sides of the facility in determining the access-point density on one-way segments.

Step 3: Select FFS Curve

As noted previously, once the multilane highway segment's FFS is determined, one of the four base speed–flow curves from Exhibit 14-2 is selected for use in the analysis. Interpolating between curves is not recommended. Criteria for selecting an appropriate curve were given in the text preceding Exhibit 14-2.

Step 4: Adjust Demand Volume

The basic speed–flow curves of Exhibit 14-2 are based on flow rates in equivalent passenger cars per hour, with the driver population dominated by regular users of the multilane highway segment. Demand volumes expressed as vehicles per hour under prevailing conditions must be converted to this basis. Equation 14-3 is used for this adjustment:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

Equation 14-3

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;

N = number of lanes (one direction);

f_{HV} = adjustment factor for presence of heavy vehicles in traffic stream, from Equation 14-4; and

f_p = adjustment factor for atypical driver populations.

PHF

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 14-3 accounts for this phenomenon.

On multilane highways, typical PHFs range from 0.75 to 0.95. Lower values are typical of lower-volume conditions. Higher values 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

A *heavy vehicle* is defined as any vehicle with more than four wheels on the ground during normal operation. Such vehicles are generally categorized as trucks, buses, or RVs. Trucks cover a wide variety of vehicles, from single-unit trucks with double rear tires to triple-unit tractor–trailer combinations. Small panel or pickup trucks with only four wheels are, however, classified as passenger cars. Buses include intercity buses, public transit buses, and school buses. Because buses are in many ways similar to single-unit trucks, both types of vehicles are considered in one category. RVs include a wide variety of vehicles from self-contained motor homes to cars and small trucks with trailers (for boats, all-terrain vehicles, or other items). The heavy-vehicle adjustment factor f_{HV} is computed by using Equation 14-4:

Equation 14-4

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where

f_{HV} = heavy-vehicle adjustment factor,

P_T = proportion of trucks and buses in traffic stream,

P_R = proportion of RVs in traffic stream,

E_T = passenger-car equivalent (PCE) of one truck or bus in traffic stream, and

E_R = PCE of one RV in traffic stream.

The adjustment factor is found in a two-step process. First, the PCE for each truck, bus, and RV 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, bus, or RV under the prevailing conditions. Second, Equation 14-4 is used to convert the PCE values to the adjustment factor.

In many cases, trucks will be the only heavy vehicle present in the traffic stream. In others, the percentage of RVs will be small compared with trucks and buses. If the ratio of trucks and buses to RVs is 5:1 or greater, all heavy vehicles may be (but do not have to be) considered to be trucks.

The effect of heavy vehicles on traffic flow depends on terrain and grade conditions as well as traffic composition. PCEs can be selected for one of three conditions:

- Extended multilane highway segments in general terrain,
- Specific upgrades, or
- Specific downgrades.

Each of these conditions is more precisely defined and discussed below.

Equivalents for General Terrain Segments

General terrain can be applied where

Grades are ≤2%,

Grades are ≤0.25 mi long, or

Grades are >2% and <3%, and are ≤0.50 mi long.

General terrain refers to extended lengths of multilane highway containing a number of upgrades and downgrades where no single grade is long enough or steep enough to have a significant impact on the operation of the overall segment. As a guideline for this determination, extended-segment analysis can be applied where no one grade of 3% or more is longer than 0.25 mi, or where no single grade between 2% and 3% is longer than 0.50 mi.

There are three categories of general terrain:

- *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 substantially below that of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length of time or at frequent intervals. *Crawl speed* is the maximum sustained speed that trucks can maintain on an extended upgrade of a given percent. If the grade is long enough, trucks will be forced to decelerate to the crawl speed, which they can maintain for extended distances. Appendix A of Chapter 11, Basic Freeway Segments, contains truck performance curves that provide truck speeds for various lengths and severities of grade. The same curves may be used for uninterrupted-flow segments on multilane highways.
- *Mountainous terrain*: 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.

Mountainous terrain is relatively rare. Generally, in segments severe enough to cause the type of operation described for mountainous terrain, there will be individual grades that are longer and steeper than the criteria for general terrain analysis.

The mountainous terrain category is rarely used, because individual grades will typically be longer and steeper than the criteria for general terrain analysis.

Exhibit 14-12 shows PCEs for trucks and buses and RVs in general terrain segments.

Exhibit 14-12
PCEs for Heavy Vehicles in General Terrain Segments

Vehicle	PCE by Type of Terrain: Level	Rolling	Mountainous
Trucks and buses, E_T	1.5	2.5	4.5
RVs, E_R	1.2	2.0	4.0

Equivalents for Specific Upgrades

Any grade between 2% and 3% and longer than 0.5 mi, or 3% or greater and longer than 0.25 mi, should be considered to be a separate segment. The analysis of such segments must consider the upgrade conditions and the downgrade conditions separately, as well as whether the grade is a single, isolated grade of constant percentage or part of a series forming a composite grade. Appendix A of Chapter 11 discusses the analysis of composite grades.

Exhibit 14-13 and Exhibit 14-14 give values of E_T and E_R for trucks and buses and for RVs, respectively. These factors vary with the percent of grade, length of grade, and the proportion of heavy vehicles in the traffic stream. Maximum values occur when there are only a few heavy vehicles in the traffic stream. The equivalents decrease as the number of heavy vehicles increases because these vehicles tend to form platoons. Because heavy vehicles have more uniform operating characteristics, fewer large gaps are created in the traffic stream when they platoon, and the impact of a single heavy vehicle in a platoon is less severe than that of a single heavy vehicle in a stream primarily composed of passenger cars. The aggregate impact of heavy vehicles on the traffic stream, however, increases as the number and percentage of heavy vehicles increase.

Exhibit 14-13
PCEs for Trucks and Buses (E_T) on Upgrades

Percent Upgrade	Length (mi)	Proportion of Trucks and Buses								
		2%	4%	5%	6%	8%	10%	15%	20%	25%
≤2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
>2 – 3	0.00 – 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.50 – 0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.75 – 1.00	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>1.00 – 1.50	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>1.50	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
>3 – 4	0.00 – 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50 – 0.75	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	>0.75 – 1.00	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	>1.00 – 1.50	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	>1.50	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
>4 – 5	0.00 – 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.50 – 0.75	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	>0.75 – 1.00	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	>1.00	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
>5 – 6	0.00 – 0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.30	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	>0.30 – 0.50	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	>0.50 – 0.75	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	>0.75 – 1.00	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	>1.00	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
>6	0.00 – 0.25	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	1.0
	>0.25 – 0.30	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	>0.30 – 0.50	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	>0.50 – 0.75	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	>0.75 – 1.00	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	>1.00	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

Note: Interpolation for percentage of trucks and buses is recommended to the nearest 0.1.

Exhibit 14-14
PCEs for RVs (E_R) on Upgrades

Percent Upgrade	Length (mi)	Proportion of RVs								
		2%	4%	5%	6%	8%	10%	15%	20%	25%
≤2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
>2 – 3	0.00 – 0.50	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.50	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
>3 – 4	0.00 – 0.25	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	>0.25 – 0.50	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	>0.50	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
>4 – 5	0.00 – 0.25	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	>0.25 – 0.50	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 0.50	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
>5	0.00 – 0.25	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	>0.25 – 0.50	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	>0.50	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0

Note: Interpolation for percentage of RVs is recommended to the nearest 0.1.

The grade length should include 25% of the length of the vertical curves at the start and end of the grade.

With two consecutive upgrades, 50% of the length of the vertical curve joining them should be included.

The length of the grade is generally taken from a highway profile. It typically includes the straight portion of the grade plus some portion of the vertical curves at the beginning and end of the grade. It is recommended that 25% of the length of the vertical curves at both ends of the grade be included in the length. Where two consecutive upgrades are present, 50% of the length of the vertical curve joining them is included in the length of each grade.

In the analysis of upgrades, the point of interest is generally at the end of the grade, where heavy vehicles have the maximum effect on operations. However, if a segment ends midgrade (because of a major access point, for example), the length of the grade to the end of the segment would be used.

The point of interest in an analysis of upgrades is usually the spot where heavy vehicles would have the greatest impact on operations: for example, the top of a grade or the top of the steepest grade in a series.

On composite grades, the relative steepness of segments is important. If a 5% upgrade is followed by a 2% upgrade, for example, the maximum impact of heavy vehicles is most likely at the end of the 5% segment. Heavy vehicles would be expected to accelerate after entering the 2% segment.

Equivalents for Specific Downgrades

Knowledge of specific impacts of heavy vehicles on operating conditions on downgrades is limited. In general, if the downgrade is not severe enough to cause trucks to shift into a lower gear (to engage engine braking), heavy vehicles may be treated as if they were on level terrain segments. Where a downgrade is severe, trucks must often use low gears to avoid gaining too much speed and running out of control. In such cases, their effect on operating conditions is more significant than on level terrain. Exhibit 14-15 gives values of E_T for this situation.

Exhibit 14-15
PCEs for Trucks and Buses (E_T) on Specific Downgrades

Percent Downgrade	Length of Grade (mi)	Proportion of Trucks and Buses			
		5%	10%	15%	20%
<4	All	1.5	1.5	1.5	1.5
4 – 5	≤4	1.5	1.5	1.5	1.5
	>4	2.0	2.0	2.0	1.5
>5 – 6	≤4	1.5	1.5	1.5	1.5
	>4	5.5	4.0	4.0	3.0
>6	≤4	1.5	1.5	1.5	1.5
	>4	7.5	6.0	5.5	4.5

On downgrades, RVs are always treated as if they were on level terrain; E_R is therefore always 1.2 on downgrades regardless of the length or severity of the downgrade or the percentage of RVs in the traffic stream.

E_R is always 1.2 on downgrades.

Equivalents for Composite Grades

The vertical alignment of most multilane highways results in a continuous series of grades. It is often necessary to determine the effect of a series of grades in succession. The most straightforward technique is to compute the *average grade,* defined as the total rise from the beginning of the composite grade to the point of interest divided by the length of the grade (to the point of interest).

The average grade technique is an acceptable approach for grades in which all subsections are less than 4% or the total length of the grade is less than 4,000 ft. For more severe composite grades, a detailed technique is presented in Appendix A of Chapter 11, Basic Freeway Segments. This technique uses vehicle performance curves and equivalent speeds to determine the equivalent simple grade for analysis. It can be applied to composite grades on multilane highways.

The average grade can be used when all component grades are <4% or the total length of the grades is <4,000 ft.

Appendix A of Chapter 11 provides a method for addressing more severe composite grades.

Adjustment for Driver Population

The base traffic stream characteristics for multilane highway segments are representative of regular drivers in a traffic stream composed substantially of commuters, or drivers who are familiar with the facility. It is generally accepted that traffic streams composed of driver populations with different characteristics (e.g., recreational drivers) use freeways less efficiently. Although data are sparse and reported results vary substantially, significantly lower capacities have been reported on weekends, particularly in recreational areas. It may generally be assumed that the reduced capacity (LOS E) extends to service flow rates and service volumes for other LOS as well.

A f_p-value of 1.00 should generally be used, reflective of drivers who are regular users of the freeway.

The adjustment factor f_p is used to reflect the effect of driver population. The values of f_p usually range from 0.85 to 1.00, although lower values have been observed in some cases. In general, the analyst should use a value of 1.00, which reflects commuters or otherwise familiar drivers, unless there is sufficient evidence that a lower value should be used. Where greater accuracy is needed, comparative field studies of commuter and recreational traffic flow and speeds are recommended.

Does LOS F Exist?

At this point, the demand flow rate has been computed and is stated in units of passenger cars per hour per lane under equivalent base conditions. This demand flow rate must be compared with the base capacity (in the same units). If demand exceeds capacity, LOS F is assigned, and the analysis ends. If demand is less than capacity, LOS F does not exist, and the analysis continues.

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 estimated speed and density of the traffic stream may be determined.

With the equations specified in Exhibit 14-3, the expected mean speed of the traffic stream can be computed. A graphical solution using Exhibit 14-2 can also be performed.

With the estimated speed determined, Equation 14-5 is used to estimate the density of the traffic stream:

Equation 14-5

$$D = \frac{v_p}{S}$$

where

D = density (pc/mi/ln),

v_p = demand flow rate (pc/h/ln), and

S = mean speed of traffic stream (mi/h).

Step 6: Determine LOS

Exhibit 14-4 is entered with the density obtained from Equation 14-5 to determine the expected prevailing LOS.

BICYCLE MODE

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 car drivers. Cyclists travel much more slowly than the prevailing traffic flow and stay as far to the right as possible, including using paved shoulders when available. This similarity indicates the need for only one model.

Follow the step-by-step description of the bicycle LOS method given in Chapter 15, Two-Lane Highways, when bicycle LOS on multilane highways is calculated.

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 14-6.

Full details on the bicycle LOS methodology and calculation procedures are given in Chapter 15, Two-Lane Highways.

3. APPLICATIONS

The analysis methodology for multilane highway segments is relatively straightforward. Thus, it can be directly used in any one of four applications:

1. *Operational analysis*: All traffic and roadway conditions are specified for an existing facility or a future facility with forecast conditions. The existing or expected LOS is determined.
2. *Design analysis*: A forecast demand volume is used, and key design parameters are specified (e.g., lane width and lateral clearance). The number of lanes required to deliver a target LOS is determined.
3. *Planning and preliminary engineering*: The basic scenario is the same as that for design analysis except that the analysis is conducted at a much earlier stage in the development process. Inputs include default values, and the demand volume is usually stated as an AADT.
4. *Service flow rates and service volumes*: The service flow rate, service volume, daily service volume, or all three are estimated for each LOS for an existing or future facility. All traffic and roadway conditions must be specified for this type of analysis.

Because the methodology and its algorithms are simple and do not involve iterations, all of the types of analysis cited can be completed without the iterative approach required by many other HCM methodologies.

DEFAULT VALUES

For this chapter's methodology, a range of input data is needed. Most of these data should be field measured or estimated values for the specific segment under consideration. When some of the data are not available, default values may be used. However, use of default values will affect the accuracy of the output. Exhibit 14-16 shows the data that are required to conduct an operational analysis and the recommended default values when site-specific data are unavailable (*4*).

Exhibit 14-16
Required Input Data and Default Values for Multilane Highway Segments

Required Data	Default Values
Geometric Data	
Number of lanes in one direction	2 or 3 (in one direction), must have site-specific value
Lane width	12 ft
TLC	12 ft
Access-point density	8 access points/mi (rural) 16 access points/mi (low-density suburban) 25 access points/mi (high-density suburban)
Terrain or specific grade (%, length)	No default, must have site-specific value
Base FFS	65 mi/h
Demand Data	
Length of analysis period	15 min
PHF	0.88, rural; 0.95, suburban
Percentage of heavy vehicles	10%, rural; 5%, urban*
Driver population factor	1.00

Note: *Alternative state-specific default values for percentage of heavy vehicles are given in Chapter 26, Freeway and Highway Segments: Supplemental.

The analyst may also replace the default values of Exhibit 14-16 with defaults that have been locally calibrated.

ESTABLISHING ANALYSIS BOUNDARIES

The methodology of this chapter applies to an uninterrupted-flow segment of multilane highway with uniform prevailing conditions. Thus, any point at which one or more of the prevailing conditions change should mark the beginning of a new analysis segment. The following conditions generally necessitate segmenting the highway:

Grade changes of 2% or more, changes in the highway's geometric characteristics, changes in speed limit, signal or STOP control of the highway, and major access points are places where multilane highways should be segmented.

- A change in the basic number of travel lanes on the highway;
- A change in the highway's median treatment;
- A change of grade of 2% or more or a constant upgrade longer than 4,000 ft;
- The presence of a traffic signal, STOP sign, or roundabout along the multilane highway;
- A significant change in the access-point density;
- A change in the speed limit;
- An access point at which a significant number or percentage of vehicles enters or leaves the highway; and
- The presence of a bottleneck condition.

In general, when analysis boundaries are established, the minimum length of a study segment should be 2,500 ft. The boundary of a study segment should be no closer than 0.25 mi to a traffic signal.

TYPES OF ANALYSIS

Operational Analysis

Operational analyses find the expected LOS for specified roadway and traffic conditions.

The operational analysis application was fully specified in the Methodology section of this chapter. Operational analysis begins with all input parameters specified and is used to find the expected LOS that would result from the prevailing roadway and traffic conditions.

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 passenger cars per hour per lane, by using Equation 14-6 instead of Equation 14-3:

$$v = \frac{V}{PHF \times f_{HV} \times f_p}$$

Equation 14-6

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 14-17. These values are selected from the base speed–flow curves of Exhibit 14-5 for each LOS.

Exhibit 14-17
Maximum Service Flow Rates (pc/h/ln) for Multilane Highway Segments Under Base Conditions

FFS	Target LOS				
(mi/h)	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

Then the number of lanes required to deliver the target LOS can be found from Equation 14-7:

Equation 14-7

$$N = \frac{v}{MSF_i}$$

where N is the number of lanes required and MSF_i is the maximum service flow rate for LOS i from Exhibit 14-17. Equation 14-6 and Equation 14-7 can be conveniently combined as Equation 14-8:

Equation 14-8

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

where all variables are as previously defined.

All fractional values of N must be rounded up.

The value of N resulting from Equation 14-7 or Equation 14-8 will most likely be fractional. Because only integer numbers of lanes can be constructed, the result is always rounded to the next higher value. Thus, if the result is 2.1 lanes, 3 lanes must be provided. In effect, the minimum number of lanes needed to provide the target LOS is 2.1. If the result were rounded to 2, a poorer LOS than the target value would result.

Because only whole lanes can be built, it may not be possible to achieve the target LOS for a given demand volume.

This rounding-up process will occasionally produce an interesting result: it is possible that a target LOS (for example, LOS C) cannot be achieved for a given demand volume. If 2.1 lanes are required to produce LOS C, providing two lanes would drop the LOS, most likely to D. However, if three lanes are provided, the LOS might actually improve to B. Thus, 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 slightly lower operating condition than that originally targeted.

Planning and Preliminary Engineering

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.

The objective of planning or preliminary engineering is to get 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 be usually 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), as shown in Equation 14-9.

$$V = DDHV = AADT \times K \times D$$

Equation 14-9

where K is the proportion of AADT occurring during the peak hour, D is the proportion of peak-hour volume traveling in the peak direction, and all other variables are as previously defined.

Chapter 3, Modal Characteristics, provides additional guidance on K- *and* D*-factors.*

Once the hourly demand volume is estimated, the methodology follows the same path as that for design analysis.

Service Flow Rates, Service Volumes, and Daily Service Volumes

This chapter's methodology can be easily manipulated to produce service flow rates, service volumes, or daily service volumes, or all three, for a multilane highway segment.

Exhibit 14-17 gives values of the maximum service flow rates MSF_i for each LOS for multilane highways of various FFSs. 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 14-10:

$$SF_i = MSF_i \times N \times f_{HV} \times f_p$$

Equation 14-10

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 14-11. 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.

$$SV_i = SF_i \times PHF$$

Equation 14-11

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 14-12:

$$MSV_i = \frac{SV_i}{K \times D}$$

Equation 14-12

where all variables are as previously defined.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 14-18 and Exhibit 14-19 are generalized daily service volume tables for multilane highway segments or facilities. They are based on a set of specified typical conditions for rural and urban multilane highways:

- Percent HV = 10% (rural), 5% (urban);
- FFS = 60 mi/h;
- PHF = 0.88 (rural), 0.95 (urban); and
- Driver population factor f_p = 1.00.

Values of rural and urban daily service volumes are provided for four-lane and six-lane highways in level and rolling terrain. A range of *K*- and *D*-factors is provided. Users should enter Exhibit 14-18 and Exhibit 14-19 with local or regional values of these factors for the appropriate size of multilane highway in the appropriate terrain.

Exhibit 14-18
Generalized Daily Service Volumes for Rural Multilane Highways (1,000 veh/day)

K-Factor	*D*-Factor	Four-Lane Highways LOS B	Four-Lane Highways LOS C	Four-Lane Highways LOS D	Four-Lane Highways LOS E	Six-Lane Highways LOS B	Six-Lane Highways LOS C	Six-Lane Highways LOS D	Six-Lane Highways LOS E
Level Terrain									
0.09	0.50	33.2	48.0	63.1	73.8	49.8	71.9	94.6	110.7
	0.55	30.2	43.6	57.4	67.1	45.3	65.4	86.0	100.6
	0.60	27.7	40.0	52.6	61.5	41.5	60.0	78.9	92.2
	0.65	25.5	36.9	48.5	56.8	38.3	55.3	72.8	85.1
0.10	0.50	29.9	43.2	56.8	66.4	44.8	64.8	85.2	99.6
	0.55	27.2	39.2	51.6	60.4	40.8	58.9	77.4	90.6
	0.60	24.9	36.0	47.3	55.3	37.4	54.0	71.0	83.0
	0.65	23.0	33.2	43.7	51.1	34.5	49.8	65.5	76.6
0.11	0.50	27.2	39.2	51.6	60.4	40.8	58.9	77.4	90.6
	0.55	24.7	35.7	46.9	54.9	37.0	53.5	70.4	82.3
	0.60	22.6	32.7	43.0	50.3	34.0	49.1	64.5	75.5
	0.65	20.9	30.2	39.7	46.4	31.3	45.3	59.6	69.7
0.12	0.50	24.9	36.0	47.3	55.3	37.4	54.0	71.0	83.0
	0.55	22.6	32.7	43.0	50.3	34.0	49.1	64.5	75.5
	0.60	20.8	30.0	39.4	46.1	31.1	45.0	59.2	69.2
	0.65	19.2	27.7	36.4	42.6	28.7	41.5	54.6	63.9
Rolling Terrain									
0.09	0.50	29.8	43.1	56.7	66.3	44.7	64.6	85.0	99.4
	0.55	27.1	39.2	51.5	60.3	40.7	58.8	77.3	90.4
	0.60	24.9	35.9	47.2	55.2	37.3	53.9	70.8	82.9
	0.65	22.9	33.1	43.6	51.0	34.4	49.7	65.4	76.5
0.10	0.50	26.8	38.8	51.0	59.7	40.3	58.2	76.5	89.5
	0.55	24.4	35.3	46.4	54.2	36.6	52.9	69.6	81.4
	0.60	22.4	32.3	42.5	49.7	33.6	48.5	63.8	74.6
	0.65	20.7	29.8	39.2	45.9	31.0	44.7	58.9	68.8
0.11	0.50	24.4	35.3	46.4	54.2	36.6	52.9	69.6	81.4
	0.55	22.2	32.0	42.2	49.3	33.3	48.1	63.2	74.0
	0.60	20.3	29.4	38.6	45.2	30.5	44.1	58.0	67.8
	0.65	18.8	27.1	35.7	41.7	28.2	40.7	53.5	62.6
0.12	0.50	22.4	32.3	42.5	49.7	33.6	52.9	63.8	74.6
	0.55	20.3	29.4	38.6	45.2	30.5	48.1	58.0	67.8
	0.60	18.6	26.9	35.4	41.4	28.0	44.1	53.1	62.1
	0.65	17.2	24.9	32.7	38.2	25.8	40.7	49.0	57.4

Note: Key assumptions: 12% trucks, 0.88 PHF, 60-mi/h FFS, driver population factor 1.0.

Exhibit 14-19
Generalized Daily Service Volumes for Urban Multilane Highways (1,000 veh/day)

K-Factor	D-Factor	Four-Lane Highways LOS B	Four-Lane Highways LOS C	Four-Lane Highways LOS D	Four-Lane Highways LOS E	Six-Lane Highways LOS B	Six-Lane Highways LOS C	Six-Lane Highways LOS D	Six-Lane Highways LOS E
Level Terrain									
0.08	0.50	48.3	69.3	88.5	98.4	72.4	104.0	132.8	147.5
	0.55	43.9	63.0	80.5	89.4	65.8	94.5	120.7	134.1
	0.60	40.2	57.8	73.8	82.0	60.4	86.6	110.7	123.0
	0.65	37.1	53.3	68.1	75.7	55.7	80.0	102.1	113.5
0.09	0.50	42.9	61.6	78.7	87.4	64.4	92.4	118.0	131.2
	0.55	39.0	56.0	71.5	79.5	58.5	84.0	107.3	119.2
	0.60	35.8	51.3	65.6	72.9	53.7	77.0	98.4	109.3
	0.65	33.0	47.4	60.5	67.3	49.5	71.1	90.8	100.9
0.10	0.50	38.6	55.4	70.8	78.7	57.9	83.2	106.2	118.0
	0.55	35.1	50.4	64.4	71.5	52.7	75.6	96.6	107.3
	0.60	32.2	46.2	59.0	65.6	48.3	69.3	88.5	98.4
	0.65	29.7	42.6	54.5	60.5	44.6	64.0	81.7	90.8
0.12	0.50	35.1	50.4	64.4	71.5	52.7	75.6	96.6	107.3
	0.55	31.9	45.8	58.5	65.0	47.9	68.7	87.8	97.6
	0.60	29.3	42.0	53.7	59.6	43.9	63.0	80.5	89.4
	0.65	27.0	38.8	49.5	55.0	40.5	58.2	74.3	82.5
Rolling Terrain									
0.08	0.50	44.8	64.4	82.2	91.3	67.3	96.5	123.3	137.0
	0.55	40.8	58.5	74.7	83.0	61.1	87.8	112.1	124.6
	0.60	37.4	53.6	68.5	76.1	56.0	80.4	102.8	114.2
	0.65	34.5	49.5	63.2	70.3	51.7	74.3	94.9	105.4
0.09	0.50	39.9	57.2	73.1	81.2	59.8	85.8	109.6	121.8
	0.55	36.2	52.0	66.4	73.8	54.4	78.0	99.6	110.7
	0.60	33.2	47.7	60.9	67.7	49.8	71.5	91.3	101.5
	0.65	30.7	44.0	56.2	62.5	46.0	66.0	84.3	93.7
0.10	0.50	35.9	51.5	65.8	73.1	53.8	77.2	98.6	109.6
	0.55	32.6	46.8	59.8	66.4	48.9	70.2	89.7	99.6
	0.60	29.9	42.9	54.8	60.9	44.8	64.4	82.2	91.3
	0.65	27.6	39.6	50.6	56.2	41.4	59.4	75.9	84.3
0.12	0.50	32.6	46.8	59.8	66.4	48.9	77.2	89.7	99.6
	0.55	29.6	42.5	54.4	60.4	44.5	70.2	81.5	90.6
	0.60	27.2	39.0	49.8	55.4	40.8	64.4	74.7	83.0
	0.65	25.1	36.0	46.0	51.1	37.6	59.4	69.0	76.6

Note: Key assumptions: 8% trucks, 0.93 PHF, 60-mi/h FFS, driver population factor 1.0.

Exhibit 14-18 and Exhibit 14-19 must be used with care. Because the characteristics of any given multilane highway may or may not be typical, the values should not be used in the analysis of a specific segment of multilane highway. The exhibits are intended to allow a general evaluation of many facilities within a given jurisdiction on a first-pass basis to identify those segments or facilities that might be in need of remediation. Any segments or facilities so identified should then be submitted to specific analysis by using this chapter's methodology and each segment's site-specific characteristics. Exhibit 14-18 and Exhibit 14-19 should not be used to make final decisions on which segments or facilities to upgrade or on the specific designs proposed for such upgrades.

Daily service volumes are computed with Equation 14-10 through Equation 14-12, which combined yield Equation 14-13:

$$DSV_i = \frac{MSF_i \times N \times f_{HV} \times f_p \times PHF}{K \times D}$$

Equation 14-13

where all variables are as previously defined. Values of *MSF* are selected from Exhibit 14-18 or Exhibit 14-19 for the typical FFS of 60 mi/h. Exhibit 14-18 and Exhibit 14-19 do not show LOS A, since this level is rarely of interest in assessing improvement programs.

For multilane highways, daily service volume tables are quite easy to construct by using localized typical values and local defaults. Equation 14-13 is easily applied. All of the variables in the equation simply have to be defined for a given FFS. The heavy-vehicle adjustment depends on PCEs, which are easily obtained for each of the terrain categories.

USE OF ALTERNATIVE TOOLS

Except for the effects of interaction with other facilities, the limitations of the methodology that were stated earlier in the chapter have minimal potential to be addressed by alternative tools. There is thus insufficient experience with alterative tools to support the development of useful guidance for their application to multilane highways.

4. EXAMPLE PROBLEMS

Exhibit 14-20
List of Example Problems

Problem Number	Description	Application
1	LOS on an undivided four-lane highway	Operational analysis
2	LOS on a five-lane highway with TWLTL	Operational analysis
3	Design cross section required to provide target LOS	Design analysis
4	Multilane highway modernization	Planning analysis
5	Future cross section required to provide target LOS	Planning analysis

EXAMPLE PROBLEM 1: LOS ON UNDIVIDED FOUR-LANE HIGHWAY

A 3.25-mi, undivided four-lane highway is primarily on level terrain. The highway does, however, contain one sustained grade of 2.5% that is 3,200 ft long. At what LOS is the highway expected to operate?

The Facts

- Level terrain; 3,200-ft, 2.5% grade included;
- Base FFS = 65 mi/h;
- Lane width: 11 ft;
- Clearance at roadside: 4 ft;
- Access points per mile: 20;
- Peak-hour volume: 1,900 veh/h;
- Traffic composition: 13% trucks, 2% RVs;
- PHF = 0.90; and
- Familiar facility users.

Comments

Three solutions will be needed in this case: (*a*) for the level terrain portion of the highway, (*b*) for the 2.5% upgrade portion of the highway, and (*c*) for the 2.5% downgrade portion of the highway. The factor that will vary for each of these is the heavy-vehicle adjustment factor.

Step 1: Input Data

All input data are specified in the example problem statement.

Step 2: Compute FFS

The FFS is estimated with Equation 14-1. The BFFS is given as 65 mi/h. Adjustments are needed for

- Lane width f_{LW} = 1.9 mi/h (Exhibit 14-8, with 11-ft lanes);
- Lateral clearance f_{LC} = 0.4 mi/h (Exhibit 14-9, with TLC = 4 + 6 = 10 ft, four lanes);
- Median type f_M = 1.6 mi/h (Exhibit 14-10, for undivided highways); and
- Access-point density f_A = 5.0 mi/h (Exhibit 14-11, with 20 access points/mi).

Then

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

$$FFS = 65.0 - 1.9 - 0.4 - 1.6 - 5.0 = 56.1 \text{ mi/h}$$

Step 3: Select FFS Curve

FFSs are all rounded to the nearest 5 mi/h. Therefore, the FFS used in the calculation will be 55 mi/h.

Step 4: Adjust Demand Volume

The demand volume, stated in vehicles per hour under prevailing conditions, must be converted to a demand flow rate in passenger cars per hour under base conditions by using Equation 14-3:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

where

V = demand volume (veh/h) (1,900 veh/h, given),

PHF = peak hour factor (0.90, given), and

f_p = driver population factor (1.00, familiar users).

The heavy-vehicle adjustment factor is computed by using Equation 14-4:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Three sets of PCEs have to be determined: for level terrain, for 2.5% upgrade, and for 2.5% downgrade.

- Level terrain: E_T = 1.5 (Exhibit 14-12); E_R = 1.2 (Exhibit 14-12);
- Upgrade: E_T = 1.5 (Exhibit 14-13, with 13% trucks and a 2.5% grade; 3,200/5,280 = 0.61 mi); E_R = 3.0 (Exhibit 14-14 with 2% RVs and 2.5% grade for 0.61 mi); and
- Downgrade: E_T = 1.5 (Exhibit 14-15 with less than 4% grade and 13% trucks); E_R = 1.2 (from the text following Exhibit 14-15).

In this case, the equivalents are the same for the level terrain segments and the 2.5% downgrade. Consequently, there are only two different heavy-vehicle adjustment factors to work with:

$$f_{HV}(\text{level, down}) = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(1.2 - 1)} = 0.935$$

$$f_{HV}(\text{up}) = \frac{1}{1 + 0.13(1.5 - 1) + 0.02(3.0 - 1)} = 0.905$$

There are thus two values of flow rate in passenger cars per hour under base conditions:

$$v_p\,(\text{level, down}) = \frac{1{,}900}{0.90 \times 2 \times 0.935 \times 1} = 1{,}129 \text{ pc/h}$$

$$v_p\,(\text{up}) = \frac{1{,}900}{0.90 \times 2 \times 0.905 \times 1} = 1{,}166 \text{ pc/h}$$

Step 5: Estimate Speed and Density

Speed for the two demand levels can be estimated by using the equations in Exhibit 14-3 or graphically by using Exhibit 14-5. In this case, both demand flow rates, 1,129 pc/h and 1,166 pc/h, are less than 1,400 pc/h. From Exhibit 14-3, the speed for both of these situations is the FFS, or 55 mi/h.

The densities of the general-terrain and specific-grade segments are then computed with Equation 14-5:

$$D = \frac{v_p}{S}$$

$$D\,(\text{level, down}) = \frac{1{,}129}{55} = 20.5 \text{ pc/mi/ln}$$

$$D\,(\text{up}) = \frac{1{,}166}{55} = 21.2 \text{ pc/mi/ln}$$

Step 6: Determine LOS

As shown in Exhibit 14-4, the LOS for both densities is C.

Discussion

The multilane highway described here operates at LOS C throughout the entire study area, including the upgrade and the downgrade within it. In a sense, this problem involves a *facility* as opposed to a *segment*. The facility contains several component segments: level-terrain segments on either side of the 2.5%, 3,200-ft grade and the uphill and downhill portions of the 2.5% grade itself.

This chapter's methodology applies to uniform multilane highway segments. In this example problem, there were three segments, which together formed a facility of more than 3 mi. LOS C on all segments is very likely acceptable and would not generally call for immediate remediation.

EXAMPLE PROBLEM 2: LOS ON FIVE-LANE HIGHWAY WITH TWLTL

An 11,000-ft segment of a five-lane highway (two travel lanes in each direction plus a TWLTL) includes a 4% grade of 6,000 ft followed by 5,000 ft of level terrain. At what LOS is the facility expected to operate?

The Facts

- Lane width: 12 ft;
- Lateral clearance, both sides of the roadway: 12 ft;

- Traffic composition: 6% trucks, 0% RVs;
- Access points per mile on the level segment: eastbound, 10; westbound, 13;
- Access points per mile on the specific grade segment: eastbound, 10; westbound, 0;
- PHF = 0.90;
- Familiar users of the facility;
- Peak-hour demand: 1,500 veh/h;
- The upgrade occurs in the westbound direction; and
- Posted speed limit = 45 mi/h.

Comments

This problem is similar to Example Problem 1 in that there are three segments in the facility as described, each of which must be analyzed. The upgrade and downgrade on the 4% grade must be separately analyzed, as well as the level terrain segment. This case is a bit more complex, since not all characteristics of the segments are the same, particularly access points. Because no BFFS is given, it will be estimated as the speed limit plus 7 mi/h, or 45 + 7 = 52 mi/h.

Step 1: Input Data

All input data are given in the example problem statement.

Step 2: Compute FFS

The FFS is estimated by using Equation 14-1:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

In this case, the BFFS is estimated to be 52 mi/h. The lane width is 12 ft, which is the base condition; therefore f_{LW} = 0.0 mi/h (Exhibit 14-8). The lateral clearance is 12 ft at each roadside, but a maximum value of 6 ft may be used. A TWLTL is considered to have a median lateral clearance of 6 ft. Thus, the TLC is 6 + 6 = 12 ft, which is also a base condition. Therefore, f_{LC} = 0.0 mi/h (Exhibit 14-9). The median type adjustment f_M is also 0.0 mi/h (Exhibit 14-10).

For this example problem, only the access-point density produces a nonzero adjustment to the BFFS. Both eastbound (EB) segments (level terrain, 4% downgrade) have 10 access points/mi. From Exhibit 14-11, the corresponding adjustment factor is 2.5 mi/h. The westbound (WB) level-terrain segment has 13 access points/mi, and an adjustment factor of 3.3 mi/h (by interpolation in Exhibit 14-11). The WB upgrade has 0 access points/mi and an adjustment factor of 0.0 mi/h. Therefore

$$FFS_{\text{EB,Level,Downgrade}} = 52.0 - 0.0 - 0.0 - 0.0 - 2.5 = 49.5 \text{ mi/h}$$

$$FFS_{\text{WB,Level}} = 52.0 - 0.0 - 0.0 - 0.0 - 3.3 = 48.7 \text{ mi/h}$$

$$FFS_{\text{WB,Upgrade}} = 52.0 - 0.0 - 0.0 - 0.0 - 0.0 = 52.0 \text{ mi/h}$$

Step 3: Select FFS Curve

Despite the fact that slight differences in FFS exist, all three segment analyses will use the 50-mi/h speed–flow curve.

Step 4: Adjust Demand Volume

Demand volume is adjusted by using Equation 14-3:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p}$$

where

V = 1,500 veh/h (given),

PHF = 0.90 (given),

N = 2 lanes (given), and

f_p = 1.00 (familiar users, given).

To compute the heavy-vehicle adjustment factor (Equation 14-4), PCEs for trucks are needed for (*a*) level terrain, (*b*) a 4%, 6,000-ft upgrade, and (*c*) a 4%, 6,000-ft downgrade. The following values are obtained:

- Level terrain: 1.5 (Exhibit 14-12);
- Upgrade: 3.0 (Exhibit 14-13, with 6% trucks and a 4% grade, 6,000/5,280 = 1.14 mi); and
- Downgrade: 1.5 (Exhibit 14-15 with a 4% grade less than 4 mi long).

As in Example Problem 1, the downgrade equivalent is the same as the equivalent for level terrain. Therefore, there are only two heavy-vehicle adjustment factors (Equation 14-4):

$$f_{HV}(\text{level,down}) = \frac{1}{1+0.06(1.5-1)} = 0.971$$

$$f_{HV}(\text{up}) = \frac{1}{1+0.06(3.0-1)} = 0.893$$

and

$$v_p(\text{level, down}) = \frac{1500}{0.90 \times 2 \times 0.971 \times 1.0} = 858 \text{ pc/mi/ln}$$

$$v_p(\text{up}) = \frac{1500}{0.90 \times 2 \times 0.893 \times 1.0} = 933 \text{ pc/mi/ln}$$

Step 5: Estimate Speed and Density

Speed is estimated by using the equations of Exhibit 14-3 or the graph in Exhibit 14-5. With the equations of Exhibit 14-3, both demand flow rates are less than 1,400 pc/h/ln. Therefore, the speeds are equal to the FFSs, both of which are 50 mi/h.

Density is computed by using Equation 14-5:

$$D\ (\text{level, down}) = \frac{858}{50} = 17.1\ \text{pc/mi/ln}$$

$$D\ (\text{up}) = \frac{933}{50} = 18.7\ \text{pc/mi/ln}$$

Step 6: Determine LOS

The LOS is found by comparing the densities of the segments with the criteria in Exhibit 14-4. The level terrain and downgrade segments operate at LOS B. The upgrade segment operates at LOS C.

Discussion

Even though the upgrade technically operates at LOS C, it is very close to the LOS B boundary (18.0 pc/mi/ln). All segments of the multilane highway facility described operate well. No remediation would likely be needed.

EXAMPLE PROBLEM 3: DESIGN CROSS SECTION REQUIRED TO PROVIDE TARGET LOS

A new 2-mi segment of multilane highway will be built within a 150-ft right-of-way. Sixty feet of right-of-way will be reserved for clear zones; therefore, 90 ft of width will be available for travel lanes, shoulders, and median. How many travel lanes are needed to provide LOS D during the peak hour?

The Facts

- AADT = 60,000 veh/day;
- $D = 0.10$; $K = 0.55$;
- 50-mi/h speed limit;
- Rolling terrain;
- Traffic composition: 5% trucks, no RVs;
- PHF = 0.90;
- Access-point density = 10.0 access points/mi; and
- Familiar facility users.

Comments

This problem is potentially iterative. The exact cross section is unknown—not only the number of lanes but also the lateral clearances and median treatment. Thus, assumptions will be made and will have to be checked when the trial analysis is complete. To begin the solution, it will be assumed that 12-ft lanes will be provided and that 6-ft clearances at the roadside and median will also be provided (TLC = 6 + 6 = 12 ft). A divided highway will also be assumed for initial computations.

Step 1: Input Data

All input data are specified in the example problem statement.

Step 2: Compute FFS

No BFFS is given. It will be assumed that the BFFS will be 5 mi/h more than the posted speed limit, or 50 + 5 = 55 mi/h. FFS is estimated by using Equation 14-1. Given that the lane width, lateral clearance, and median treatments assumed are all base conditions, there is no adjustment for these. The only adjustment is for access-point density. On the basis of Exhibit 14-11 for 10 access points/mi, the adjustment is 2.5 mi/h. Therefore

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

$$FFS = 55.0 - 0.0 - 0.0 - 0.0 - 2.5 = 52.5 \text{ mi/h}$$

Step 3: Select FFS Curve

Following this chapter's guidelines, for a FFS of 52.5 mi/h, the 55-mi/h base speed–flow curve will be used in this analysis.

Step 4: Determine Number of Lanes Needed for LOS D

Step 4 has a number of intermediate computations. First, the demand volume is stated as an AADT. This volume must be converted to an estimated directional design-hour volume ($V = DDHV$) by using Equation 14-9:

$$V = DDHV = AADT \times K \times D$$

$$V = DDHV = 60{,}000 \times 0.10 \times 0.55 = 3{,}300 \text{ veh/h}$$

The number of lanes required to meet a target LOS is estimated with Equation 14-8:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

where

MSF = 1,850 pc/h/ln (Exhibit 14-17, with LOS D and 55 mi/h);

PHF = 0.90 (given); and

f_p = 1.00 (familiar users).

The heavy-vehicle adjustment factor is estimated by using Equation 14-4 and the PCE for trucks in rolling terrain from Exhibit 14-12, which is 2.5. Then

$$f_{HV} = \frac{1}{1 + 0.05(2.5 - 1)} = 0.930$$

and

$$N = \frac{3{,}300}{1{,}850 \times 0.90 \times 0.93 \times 1.00} = 2.13 \text{ lanes}$$

This result means that to meet the criteria for LOS D within the peak hour, three lanes in each direction will have to be provided and the facility will be a six-lane multilane highway.

It is necessary to consider whether the assumed cross section (now known to be six lanes) can fit in the 90-ft available width. Lane widths are 12 ft, with 6-ft lateral clearances at both roadsides and in the median. If a 12-ft median is assumed, the total width becomes (6 × 12) + (6 × 2) + 12 = 72 + 12 + 12 = 96 ft, which is in excess of the 90-ft right-of-way.

The median treatment could be reconsidered. It is not necessary to have a 12-ft median with 6-ft clearances to the inside edges of the travel pavement to produce the desired operation. A concrete median barrier with 2-ft buffers on each side would occupy a total of only 6 ft and would not be expected to have any impact on driver behavior or the FFS. Then the total width required would be (6 × 12) + (6 × 2) + 6 = 90 ft. This is the design cross section. None of the calculations done to this point would be altered by this design.

It is likely that providing a six-lane highway will result in better operations than the minimums of LOS D. With the number of lanes known, the demand flow rate under base conditions can be computed with Equation 14-3:

$$v_p = \frac{3{,}300}{0.90 \times 3 \times 0.93 \times 1.00} = 1{,}314 \text{ pc/h/ln}$$

Step 5: Estimate Speed and Density

The speed of the traffic stream can be determined by using the equations of Exhibit 14-3 or the graph in Exhibit 14-5. On the basis of the equations, because the demand flow rate is less than 1,400 pc/h/ln, the speed is the FFS, or 55 mi/h in this case.

The density may now be computed by using Equation 14-5:

$$D = \frac{1{,}314}{55} = 23.9 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From the criteria of Exhibit 14-4, the LOS provided is C, one grade better than the design target of LOS D.

Discussion

The design resulted in a six-lane cross section on a divided multilane highway with no clearance obstructions. The LOS provided is better than the design target, since three lanes were provided in each direction while only 2.13 lanes were necessary.

EXAMPLE PROBLEM 4: MULTILANE HIGHWAY MODERNIZATION

A 2.5-mi segment of a substandard multilane highway is to be improved by providing wider shoulders, widening the lanes to 12 ft, improving the alignment on a few sharp curves, restricting the number of roadside access points, and adding a median. These improvements will increase the FFS of the facility from 50 mi/h to 60 mi/h. How much additional traffic can be accommodated while the postimprovement LOS is maintained?

The Facts

- Demand flow rate = 1,400 pc/h/ln under base conditions.

Comments

This problem is relatively straightforward. Most of the steps in a standard analysis can be skipped, since the present and future FFS are given, and the demand flow rate has already been reduced to base conditions.

Step 1: Find Existing LOS

With the equations of Exhibit 14-3, the speed of vehicles in the existing configuration will be the FFS, or 50 mi/h. With Equation 14-5, the density is computed as

$$D = \frac{1{,}400}{50} = 28 \text{ pc/mi/ln}$$

From Exhibit 14-4, this is LOS D.

Step 2: Find Expected LOS After Improvement

With the equations of Exhibit 14-3, the speed of vehicles on the improved cross section will also be the FFS, or 60 mi/h. The density is computed with Equation 14-5:

$$D = \frac{1{,}400}{60} = 23.2 \text{ pc/mi/ln}$$

From Exhibit 14-4, this is LOS C.

Step 3: Find Additional Volume Under LOS C

From Exhibit 14-5, the maximum service flow rate for LOS C on a 60-mi/h multilane highway is 1,550 pc/h/ln. The existing demand flow rate is 1,400 pc/h/ln. The additional demand flow possible while LOS C is maintained is 1,550 – 1,400 = 150 pc/h/ln. Since there are three lanes in each direction, the demand flow rate can increase by 3 × 150 = 450 pc/h without slipping into LOS D.

Discussion

This example problem illustrates how the methodology can be adapted to different uses, in this case, evaluating the impact of a proposed improvement to a multilane highway. The LOS improves from D to C, and an additional peak-hour demand flow rate of 450 pc/h can be accommodated by the improved highway while LOS C is maintained.

EXAMPLE PROBLEM 5: FUTURE CROSS SECTION REQUIRED TO PROVIDE TARGET LOS

A new suburban multilane highway is being planned. The opening-day forecast AADT is 42,000 vehicles per day. How many lanes will be needed to provide for LOS C during the peak hour on opening day?

The Facts

Since this is a planning application, many details cannot be based on current information. As a result, some of the "facts" are forecasts, and default values based on regional data are used to complete the list of facts needed for the analysis.

- Demand = 42,000 veh/day;
- $K = 0.10$; $D = 0.60$;
- Traffic composition: 10% trucks, no RVs;
- Rolling terrain;
- Base FFS = 55 mi/h;
- Lane width: 12 ft; roadside lateral clearance: 6 ft;
- Undivided highway;
- Access-point density = 6 access points/mi;
- PHF = 0.90; and
- Commuter traffic.

Comments

The demand volume, given as an AADT, must be converted to a DDHV. Once this is done, the example becomes a design application to determine the number of lanes needed to deliver LOS C.

Step 1: Input Data

All input data for this problem are specified in the problem statement.

Step 2: Compute FFS

The FFS is computed by using Equation 14-1. The BFFS is given. Lane widths and lateral clearances conform to base conditions, and no adjustments will be necessary. Then

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

where

$BFFS$ = 55 mi/h (given);

f_{LW} = 0.0 mi/h (Exhibit 14-8, with 12-ft lanes);

f_{LC} = 0.0 mi/h (Exhibit 14-9, with 6-ft clearances);

f_M = 1.6 mi/h (Exhibit 14-10, undivided);

f_A = 1.5 mi/h (Exhibit 14-11, with 6 access points/mi, interpolated);

and

$$FFS = 55.0 - 0.0 - 0.0 - 1.6 - 1.5 = 51.9 \text{ mi/h}$$

Step 3: Select FFS Curve

On the basis of criteria given in the methodology section, the 50-mi/h FFS curve will be used for this solution.

Step 4: Determine Number of Lanes Needed to Provide LOS C

The number of lanes needed to deliver LOS C on opening day is estimated with Equation 14-8:

$$N = \frac{V}{MSF_i \times PHF \times f_{HV} \times f_p}$$

The demand volume must be converted to an hourly basis by using Equation 14-9:

$$V = DDHV = AADT \times K \times D$$

$$V = DDHV = 42{,}000 \times 0.10 \times 0.60 = 2{,}520 \text{ veh/h}$$

The value of MSF is selected from Exhibit 14-17 for a 50-mi/h highway and LOS C: 1,300 pc/h. The heavy-vehicle adjustment factor f_{HV} is computed by using Equation 14-4 with a PCE of 2.5 selected from Exhibit 14-12 for trucks in rolling terrain:

$$f_{HV} = \frac{1}{1 + 0.10(2.5 - 1)} = 0.870$$

Because the demand volume is composed primarily of commuters, the adjustment factor for driver population f_p is 1.00. The PHF was given as 0.90. Then

$$N = \frac{2{,}520}{1{,}300 \times 0.90 \times 0.87 \times 1.00} = 2.48 \text{ lanes}$$

This result implies that a six-lane cross section will have to be provided. Because this cross section is more than the minimum computed, the actual demand flow rate under base conditions should be computed by using Equation 14-3:

$$v_p = \frac{2{,}520}{0.90 \times 3 \times 0.87 \times 1.00} = 1{,}073 \text{ pc/mi/ln}$$

Step 5: Estimate Speed and Density

From the equations of Exhibit 14-3, the expected speed for the demand flow rate is the FFS of 50 mi/h. The density can now be computed with Equation 14-5:

$$D = \frac{1{,}073}{50} = 21.5 \text{ pc/mi/ln}$$

Step 6: Determine LOS

From Exhibit 14-4, the six-lane multilane highway will be expected to operate at LOS C, which was the design objective.

Discussion

In this case, the target LOS has been achieved with the six-lane cross section. The highway will, however, operate in the better portion of LOS C instead of at the boundary.

5. REFERENCES

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

1. Reilly, W., D. Harwood, J. Schoen, and M. Holling. *Capacity and LOS Procedures for Rural and Urban Multilane Highways*. NCHRP Project 3-33, Final Report. JHK & Associates, Tucson, Ariz., May 1990.
2. Landis, B. W., V. R. Vattikuti, and M. T. Brannick. Real-Time Human Perceptions: Toward a Bicycle Level of Service. In *Transportation Research Record 1578*, Transportation Research Board, National Research Council, Washington, D.C., 1997, pp. 119–126.
3. Robertson, H. D. (ed.). *Manual of Traffic Engineering Studies*. Institute of Transportation Engineers, Washington, D.C., 2000.
4. Zegeer, J. D., M. A. Vandehey, M. Blogg, K. Nguyen, and M. Ereti. *NCHRP Report 599: Default Values for Highway Capacity and Level of Service Analyses*. Transportation Research Board of the National Academies, Washington, D.C., 2008.

CHAPTER 15
TWO-LANE HIGHWAYS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Two-lane highways have one lane for the use of traffic in each direction. The principal characteristic that separates motor vehicle traffic on two-lane highways from 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.

Because passing capacity decreases as passing demand increases, two-lane highways exhibit a unique characteristic: operating quality often decreases precipitously as demand flow increases, and operations can become "unacceptable" at relatively low volume-to-capacity ratios. For this reason, few two-lane highways ever operate at flow rates approaching capacity; in most cases, poor operating quality has led to improvements or reconstruction long before capacity demand is reached.

The quality of service for bicycles is primarily affected by the speed and volume of adjacent traffic flows and by the degree of separation between bicyclist and motor vehicle traffic allowed by the roadway geometry.

Chapter 15, Two-Lane Highways, presents methodologies for the analysis, design, and planning of two-lane highway facilities operating under uninterrupted flow, for both automobiles and bicycles. Uninterrupted flow exists when there are no traffic control devices that interrupt traffic and where no platoons are formed by upstream signals. In general, any segment that is 2.0 to 3.0 mi from the nearest signalized intersection would fit into this category. Where signalized intersections 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 17, Urban Street Segments, which are located in Volume 3. It is assumed that no passing in the opposing lane occurs on urban streets.

Chapter 15 also includes a methodology for predicting the effect of passing and truck climbing lanes on two-lane highways.

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 different geographical areas and serve a wide 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.

The functions of two-lane highways include efficient mobility, accessibility, scenic and recreational enjoyment, and service to small towns and communities.

Efficient mobility is the principal function of major two-lane highways that connect major trip generators or that 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 the formation of platoons, 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. Passing delays, however, 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 serve a variety of rural areas, as well as more developed areas. Therefore, this chapter's methodology and level of service (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 automobile methodology 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*).

The third class addresses two-lane highways in *developed areas*. The analysis approach for these highways is a modification of the rural highway method noted previously and was developed by the Florida Department of Transportation (FDOT) (*3*). This modification has not been subjected to a national calibration study and is based on the procedure developed and adopted by FDOT. It is presented here as an alternative procedure, since it is based entirely on information collected in Florida. For clarity, however, the material is integrated into the overall presentation and is not discussed separately as an alternative procedure.

The three classes of two-lane highways are defined as follows:

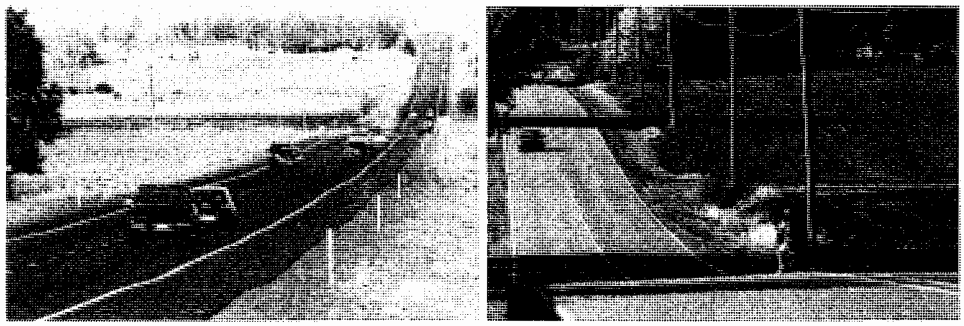
- *Class I two-lane highways* are highways where motorists expect to travel at relatively high speeds. Two-lane highways that are major intercity routes, primary connectors of major traffic generators, daily commuter routes, or major links in state or national highway networks are generally assigned 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 functioning as access routes to Class I facilities, serving as scenic or recreational routes (and not as primary arterials), or passing through rugged terrain (where high-speed operation would be impossible) are assigned to Class II. Class II 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* are highways serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns or developed recreational areas. On such segments, local traffic often mixes with through traffic, and the density of unsignalized roadside access points is noticeably higher than in a purely rural area. Class III highways may also be longer 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.

Exhibit 15-1 shows examples of the three classes of two-lane highway.

The definition of two-lane highway classes is based on their function. Most arterials or trunk roads are considered to be Class I highways, while most collectors and local roads are considered to be Class II or Class III highways. The primary determinant of a facility's classification is the motorist's expectation, which might not agree with the overall functional category of the route. 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.

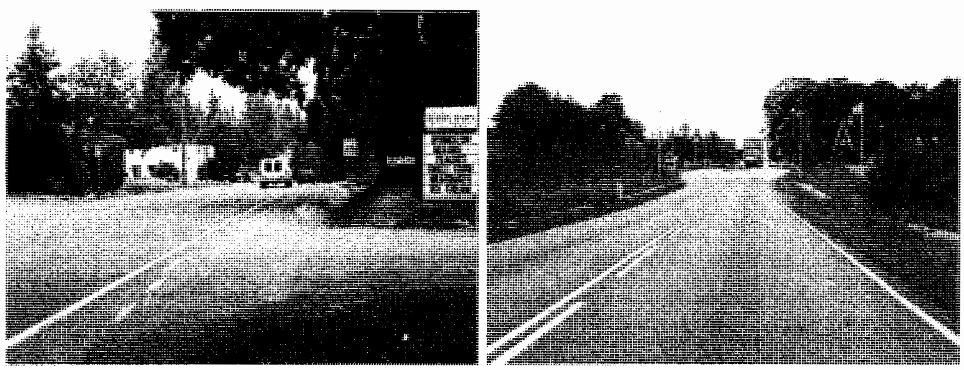
Even Class III highways incorporate only uninterrupted-flow segments of two-lane highways. Occasional signalized or unsignalized intersections on any two-lane highway must be separately analyzed with the appropriate *Highway Capacity Manual* (HCM) methodologies in Chapter 18, Signalized Intersections, Chapter 20, All-Way STOP-Controlled Intersections, or Chapter 21, Roundabouts. The results must be carefully considered in conjunction with those of uninterrupted-flow portions of the facility to obtain a complete picture of probable operations.

Exhibit 15-1
Two-Lane Highway Classification 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). The methodology of this chapter 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 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, and they may increase percent time-spent-following (PTSF) as well.

The length and frequency of no-passing zones are a result of the roadway alignment. No-passing zones may be marked by barrier centerlines in one or both directions, but any segment with a passing sight distance of less than 1,000 ft should also be considered to be a no-passing zone.

On a two-lane highway, passing in the opposing lane of flow may be necessary. 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 Relationships

Exhibit 15-2 shows the relationships among flow rate, average travel speed (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), the methodology of this chapter analyzes each direction separately.

Exhibit 15-2(b) illustrates a critical characteristic that affects two-lane highways. Low directional volumes create high values of PTSF. With only 800 pc/h, PTSF ranges from 60% (with 200 pc/h opposing flow) to almost 80% (with 1,600 pc/h opposing flow).

In multilane uninterrupted flow, typically acceptable speeds can be maintained at relatively high proportions of capacity. On two-lane highways, service quality (as measured by PTSF) begins to deteriorate at relatively low demand flows.

CAPACITY AND LOS

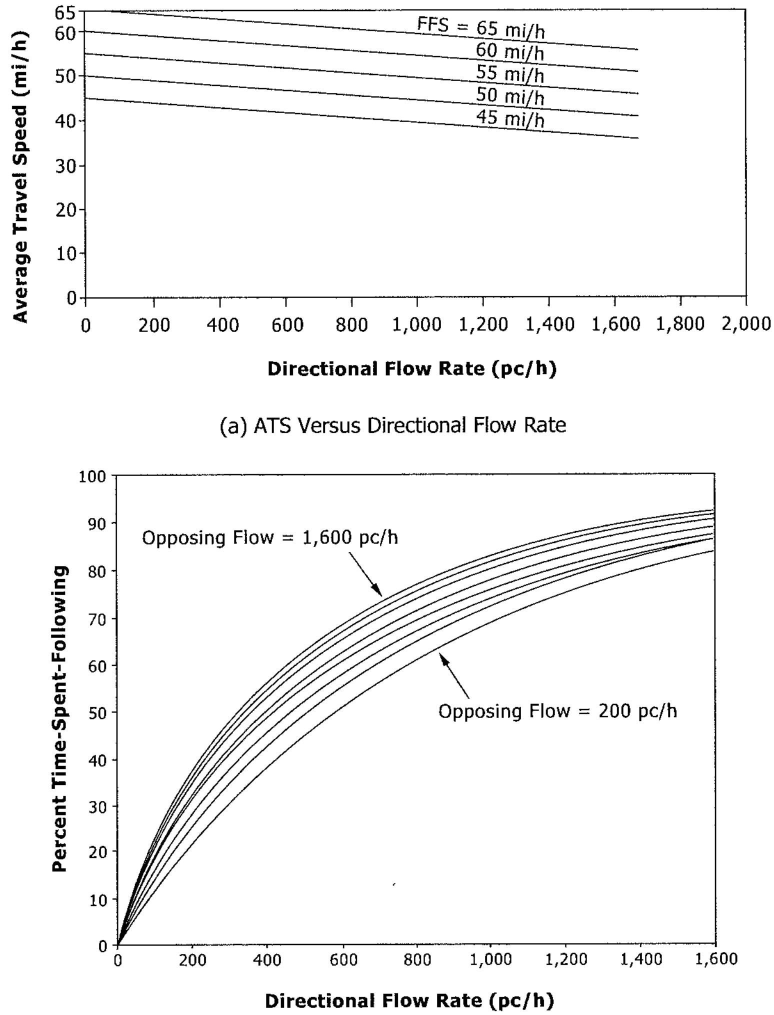
Capacity

The capacity of a two-lane highway under base conditions is 1,700 pc/h in one direction, with a limit of 3,200 pc/h for the total of the two 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 would be limited to 1,500 pc/h.

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.

Capacity conditions, however, 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.

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 important for evacuation and special event planning.

However, estimation of capacity conditions is important for evacuation planning, special event planning, and evaluation of the downstream impacts of incident bottlenecks once 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.

Capacity is not defined for bicycles on two-lane highways because of 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

Automobile Mode

Because of the wide range of situations in which two-lane highways are found, three measures of effectiveness are incorporated into the methodology of this chapter to determine automobile LOS.

1. *ATS* reflects mobility on a two-lane highway. It is defined as the highway segment length divided by the average travel time taken by vehicles to traverse it during a designated time interval.
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.

On Class I two-lane highways, speed and delay due to passing restrictions are both important to motorists. Therefore, on these highways, LOS is defined in terms of both ATS and PTSF. On Class II highways, travel speed is not a significant issue to drivers. Therefore, on these highways, LOS is defined in terms of PTSF only. On Class III highways, high speeds are not expected. Because the length of Class III segments is generally limited, passing restrictions are also not a major concern. In these cases, drivers would like to make steady progress at or near the speed limit. Therefore, on these highways, PFFS is used to define LOS. The LOS criteria for two-lane highways are shown in Exhibit 15-3.

Exhibit 15-3
Automobile LOS for Two-Lane Highways

LOS	Class I Highways		Class II Highways	Class III Highways
	ATS (mi/h)	PTSF (%)	PTSF (%)	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

Because driver expectations and operating characteristics on the three categories of two-lane highways are quite different, it is difficult to provide a single definition of operating conditions at each LOS.

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. If flow in both directions increases, a difficult trend is established: as passing demand increases, passing capacity 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 would be controlled primarily by roadway conditions. A small amount of platooning would be expected. On Class III highways, drivers should be able to maintain operating speeds close or equal to the free-flow speed (FFS) of the facility.

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, it becomes difficult to maintain FFS operation, but the speed reduction is still relatively small.

At LOS C, most vehicles are traveling 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 II facilities, but passing capacity approaches zero. A high percentage vehicles are now traveling in platoons, and PTSF is quite noticeable. On Class III highways, the fall-off from FFS is now 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 this LOS represents capacity.

LOS F exists whenever demand flow in one or both directions exceeds the capacity of the segment. 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 highways are given in Exhibit 15-4. The same LOS score is used for multilane highways, as described in Chapter 14.

Exhibit 15-4
Bicycle LOS for Two-Lane 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

REQUIRED INPUT DATA AND DEFAULT VALUES

Exhibit 15-5 lists the information necessary to apply the methodology. It also contains suggested default values for use when segment-specific information is not available. The user is cautioned, however, that every use of a default value instead of a field-measured, segment-specific variable makes the analysis results more approximate and less related to the specific conditions that describe the study site. Defaults should be used only when field measurements cannot be collected.

Exhibit 15-5
Required Input Data and Default Values for Two-Lane Highways

Required Data	Recommended Default Value	Relevant Modes
	Geometric Data	
Highway class	Must select as appropriate	Auto
Lane width	12 ft	Auto, bicycle
Shoulder width	6 ft	Auto, bicycle
Access-point density (one side)	Classes I and II: 8/mi, Class III: 16/mi	Auto
Terrain	Level or rolling	Auto
Percent no-passing zone[a]	Level: 20%, rolling: 40%, more extreme: 80%	Auto
Speed limit	Speed limit	Bicycle
Base design speed	Speed limit + 10 mi/h	Auto
Length of passing lane (if present)	Must be site-specific	Auto
Pavement condition	4 on FHWA 5-point rating scale (good)	Bicycle
	Demand Data	
Hourly automobile volume	Must be site-specific	Auto, bicycle
Length of analysis period	15 min (0.25 h)	Auto, bicycle
Peak hour factor	0.88	Auto, bicycle
Directional split	60/40	Auto, bicycle
Heavy vehicle percentage[b]	6% trucks	Auto, bicycle
Percent occupied on-highway parking	0%	Bicycle

Notes: [a] Percent no-passing zone may be different in each direction.
[b] See Chapter 26 in Volume 4 for state-specific default heavy vehicle percentages.

The use of some default values is less problematic than others. Lane and shoulder widths of 12 and 6 ft, respectively, are common, particularly on Class I highways. However, these variables have large impacts on bicycle LOS, increasing the importance of segment-specific data. A general assessment of terrain is usually straightforward and requires only general knowledge of the area through which the highway is built. Access-point densities are more difficult and tend to vary widely on a site-by-site basis. Estimating the percent no-passing zones on the basis of a generalized assessment of terrain is also challenging, since the details of vertical and horizontal alignment can have a significant impact on this factor.

FFS is best measured at the site or at a similar site. While adjustments to a base free-flow speed (BFFS) are provided as part of the methodology, no firm guidance on determining the BFFS is given. The default suggestions of Exhibit 15-5 are highly approximate.

In terms of demand data, the length of the analysis period is a recommended HCM standard of 15 min (although longer periods can be examined). The peak hour factor (PHF) is typical but could vary significantly on the basis of localized trip generation characteristics. The directional split is best observed directly, since it can vary widely over time, even at the same location. The recommended default for heavy vehicle presence is also highly approximate. This factor varies widely with local conditions; Chapter 26, Freeway and Highway Segments: Supplemental, provides state-specific default values (*4*).

As is the case with all default values, these values should be used with care, and only when site-specific data cannot be acquired by any reasonable means.

DEMAND VOLUMES AND FLOW RATES

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 the PHF to convert to flow rates is unnecessary. The worst 15-min period is selected, and flow rates are the 15-min volumes multiplied by 4. When this is done, the PHF is set at 1.00 for the rest of the application.

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 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.

2. METHODOLOGY

This section presents the details of the methodology for two-lane highways and documents its use in planning and operational analysis applications.

SCOPE OF THE METHODOLOGY

This chapter presents an operational analysis methodology for directional segments of two-lane highways for automobiles and bicyclists. Both directions may be analyzed separately on the facility or segment to obtain a full estimate of operating conditions.

This chapter's automobile methodology addresses the analysis of

- Directional segments in general terrain (level or rolling),
- Directional segments on specific grades, and
- Directional segments including passing and truck climbing lanes.

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 methodology is most directly used to determine the LOS on a uniform directional segment of two-lane highway by estimating the measures of effectiveness 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.

This chapter includes an appendix that addresses specialized treatments for two-lane highways that cannot be evaluated with the basic methodology. Special procedures are also provided to determine the impact of passing lanes or truck climbing lanes in two-lane highway segments.

LIMITATIONS OF THE METHODOLOGY

The operational analysis methodologies in this chapter do not address two-lane highways with signalized intersections. Isolated signalized intersections on two-lane highways may be evaluated with the methodology of Chapter 18, Signalized Intersections. Two-lane highways in urban and suburban areas with multiple signalized intersections 2 mi or less apart should be analyzed as urban streets or arterials with the methodology of Chapter 17, Urban Street Segments.

The bicycle 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 (*5*) 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: 45 to 50 mi/h;
- Heavy vehicle percentage: 0% to 2%; and
- Pavement condition: 1 to 5 on the Federal Highway Administration (FHWA) 5-point pavement rating scale.

The bicycle LOS methodology also 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.

AUTOMOBILE MODE

Overview

Exhibit 15-6 illustrates the basic steps in the methodology for two-lane highways. Because the three classes of highways use different service measures to determine LOS, not all steps are applied to each class of facility.

Note that the computational step for estimating ATS applies only to Class I and Class III highways, while the step for estimating PTSF applies only to Class I and Class II highways. The step for estimating PFFS applies only to Class III highways.

Segments for Analysis

The methodology of this chapter 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.

Uniform segments have the same or similar traffic and roadway conditions. Segment boundaries should be established at points where a change occurs in any of the following: terrain, lane widths or shoulder width, facility classification, or demand flow rate.

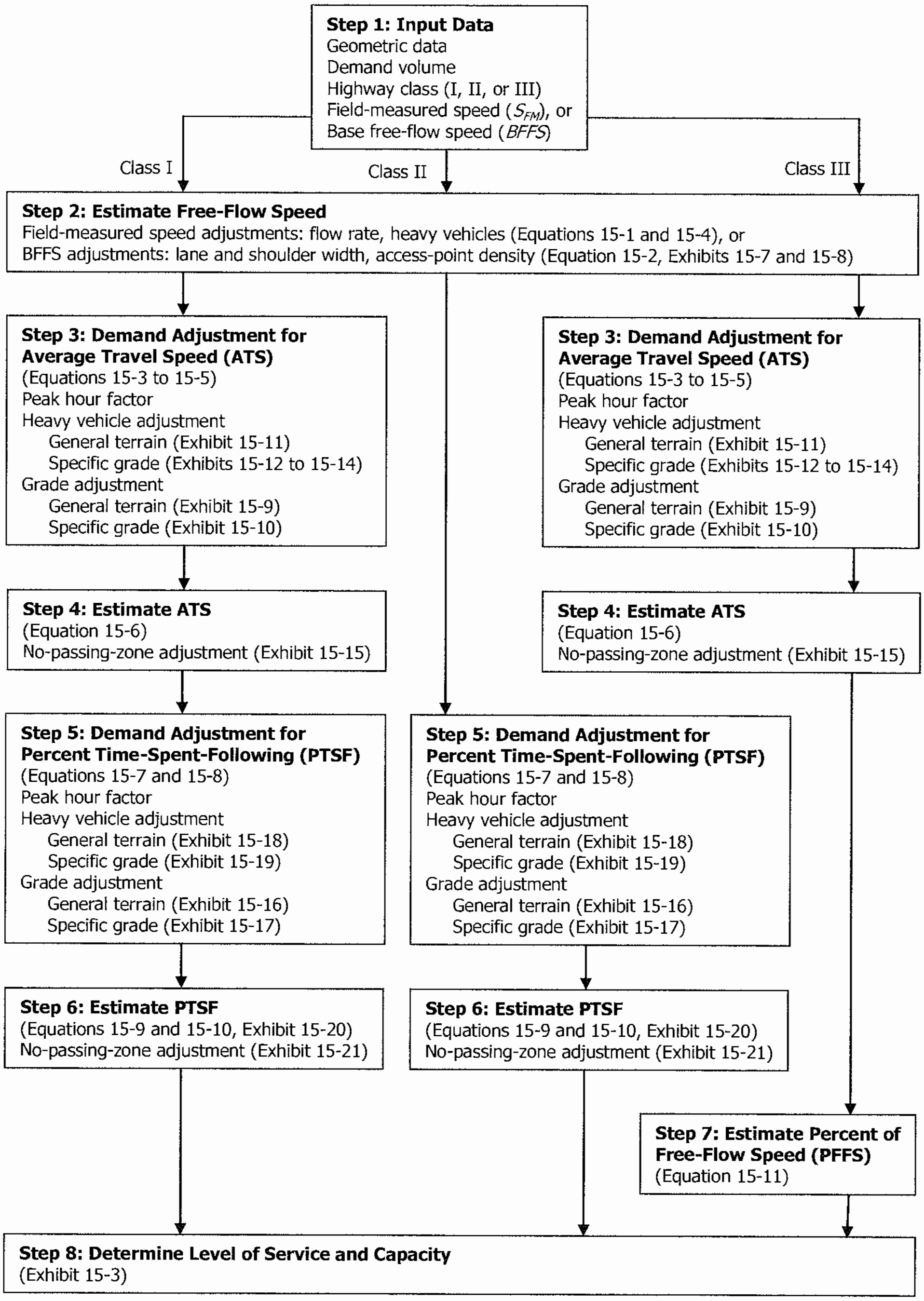


Exhibit 15-6
Flowchart of the Two-Lane Highway Methodology

Computational Steps

Step 1: Input Data

Exhibit 15-5 lists the information that must be available before a two-lane highway segment can be analyzed. The exhibit also lists default values suggested for use when site-specific data are not available.

Step 2: Estimate the FFS

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, then separate measurements in each direction are made. Each directional measurement should be based on a random 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, then measurements from a similar facility (same highway class, same speed limit, similar environment, etc.) may be used.

Field Measurements at Higher Flow Rates

For some highways, it may be difficult or impossible to observe total flow rates less than 200 veh/h. 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

FFS on two-lane highways ranges from 45 mi/h to as high as 70 mi/h. BFFS reflects alignment of the facility and the nature of traffic.

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 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 that influence 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 actual FFS may be estimated as follows:

Equation 15-2

$$FFS = BFFS - f_{LS} - f_A$$

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).

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).

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).

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 (Two Directions)	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.

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, the FFS will be the same in *both* directions. If the FFS is measured in the field, the value could be different in each direction.

If a highway contains sharp horizontal curves with design speeds substantially below those of the rest of the segment, it may be desirable to determine the FFS separately for curves and tangents and to compute a weighted-average FFS for the segment as a whole.

The data for FFS relationships in this chapter include both commuter and noncommuter traffic. There were no significant differences between the two. However, it is expected that commuters and other regular users will use a facility 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 applied only in cases of Class I and Class III two-lane highways. LOS on Class II highways is not based on ATS, and therefore this step is skipped for those highways.

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. If flow rates for the peak 15 min have been directly measured, the PHF used in Equation 15-3 is set equal to 1.00.

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 of 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, however, 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.

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.

In general, a heavy vehicle is defined as any vehicle (or vehicle–trailer unit) with more than four wheels on the ground during normal operation. Heavy vehicles are classified as trucks or recreational vehicles (RVs). Trucks cover a wide variety of vehicles from small pickup and panel trucks with more than four wheels to double and triple tractor–trailer units. Small pickup and panel trucks with only four wheels are classified as passenger cars. All school, transit, or intercity buses are classified as trucks. The RV classification also covers a wide range of vehicles, including motorized campers, motor homes, and cars or small trucks that are towing trailers.

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.

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.

Determining the heavy vehicle adjustment factor is a two-step process:

1. Passenger car equivalents are found for trucks (E_T) and RVs (E_R) under prevailing conditions.
2. A heavy vehicle adjustment factor is computed from the passenger car equivalents with Equation 15-4:

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

Equation 15-4

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 from Exhibit 15-11 or Exhibit 15-12, and

E_R = passenger car equivalent for RVs from Exhibit 15-11 or Exhibit 15-13.

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 in the next section to evaluate specific downgrades on which significant numbers of trucks must reduce their speed to crawl speed to maintain control.

Exhibit 15-12 and Exhibit 15-13 show passenger car equivalents for trucks and RVs, respectively, on specific upgrades.

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. Some downgrades, however, 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.

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	2.3	2.0
	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	1.6
	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.

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 applies only to Class I and Class III two-lane highways. Class II highways do not use ATS as a LOS measure.

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. At this point in the computational process, fully adjusted demand flow rates are available 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 is skipped for those highways.

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 will be different from those used in estimating ATS. Therefore, a detailed discussion of the process is not included here, since it is the same as that described for ATS estimates.

Equation 15-7 and Equation 15-8 are used to determine demand flow rates for the estimation of PTSF:

Equation 15-7

$$v_{i,PTSF} = \frac{V_i}{PHF \times f_{g,PTSF} \times f_{HV,PTSF}}$$

Equation 15-8

$$f_{HV,PTSF} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

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, from Exhibit 15-18 or Exhibit 15-19.

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 recreational vehicles (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 applied to Class I and Class II two-lane highways. Class III highways do not use PTSF to determine LOS.

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, from Exhibit 15-21;

$v_{d,PTSF}$ = 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 15-20 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. Equation 15-9, however, adjusts the factor to reflect the balance of flows in the analysis and opposing directions.

Step 7: Estimate the PFFS

This step is included only in the analysis of Class III two-lane highways. PFFS is not used in the determination of LOS for Class I or Class II facilities. 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 measure of effectiveness. 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.

Equation 15-12

$$c_{dATS} = 1{,}700 f_{g,ATS} f_{HV,ATS}$$

Equation 15-13

$$c_{dPTSF} = 1{,}700 f_{g,PTSF} f_{HV,PTSF}$$

where

c_{dATS} = capacity in the analysis direction under prevailing conditions based on ATS (pc/h), and

c_{dPTSF} = capacity in the analysis direction under prevailing conditions based on PTSF (pc/h).

For Class I highways, both capacities must be computed. The lower value represents capacity. For Class II highways, only the PTSF-based capacity is computed. For Class III highways, only the ATS-based capacity is computed.

One complication is that the adjustment factors depend on the demand flow rate (in vehicles per hour). Thus, adjustment factors for a base flow rate of 1,700 pc/h must be used. Technically, this value should be adjusted to reflect grade and heavy vehicle adjustments. This would create an iterative process in which a result is guessed and then checked.

In practical terms, this is unnecessary, since the highest flow group in all adjustment exhibits is greater than 900 veh/h. It is highly unlikely that any adjustments would reduce 1,700 pc/h to less than 900 veh/h. Therefore, in capacity determinations, all adjustment factors should be based on a flow rate greater than 900 veh/h.

Capacity may be limited by the directional distribution of traffic and the total two-way base capacity of 3,200 pc/h.

Another characteristic of this methodology must be considered in evaluating capacity. When the directional distribution is other than 50/50 (in level and rolling terrain), the two-way capacity implied by each directional capacity may be different. Moreover, the implied two-way capacity from either or both directions may be more than the limit of 3,200 pc/h. In such cases, the directional capacities estimated are not achievable with the stated directional distribution. If this is the case, then base capacity is restricted to 1,700 pc/h in the direction with the heaviest flow, and capacity in the opposing direction is found by using the opposing proportion of flow, with an upper limit of 1,500 pc/h.

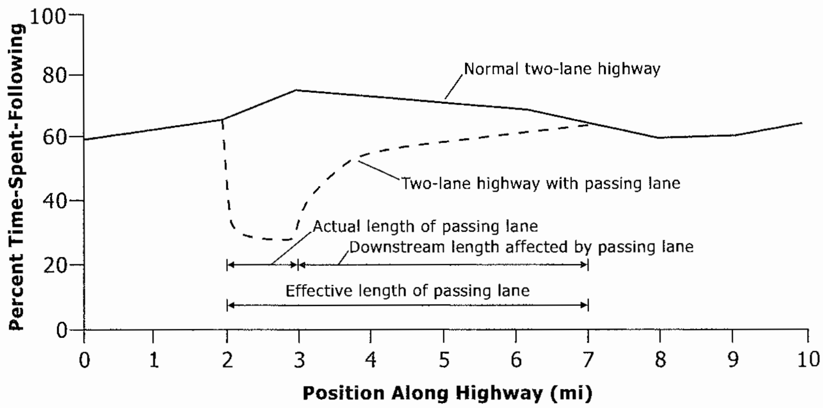
Directional Segments with Passing Lanes

Providing a passing lane on a two-lane highway in level or rolling terrain improves operational performance and therefore may improve LOS. A procedure to estimate this effect is described in this section.

This procedure should be applied only in level and rolling terrain. On specific grades, added lanes are considered to be *climbing lanes*, which are addressed in the next 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.



Source: Harwood and Hoban (*6*).

Exhibit 15-22
Operational Effect of a Passing Lane on PTSF

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, however, the effect can be far longer than the passing lane itself—up to 13 mi for low demand flow rates.

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.

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. It is recommended that the analysis segment include the full length of the passing lane's downstream effect.

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. Care must be taken, however, 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 conduct the basic analysis steps described previously. 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,
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, as 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 15-14:

Equation 15-14

$$L_d = L_t - \left(L_u + L_{pl} + L_{de}\right)$$

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 normal 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.

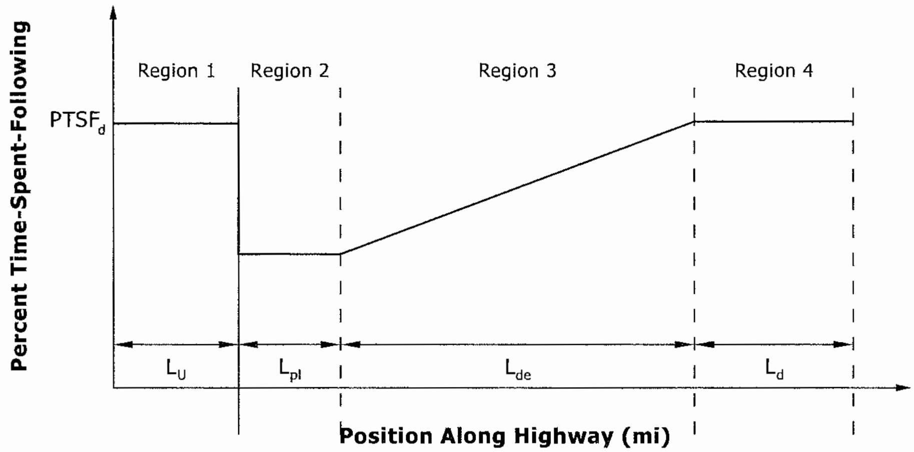


Exhibit 15-25
Effect of a Passing Lane on PTSF

On the basis of this model, the PTSF for the entire analysis segment, as affected by the passing lane, is given by Equation 15-15:

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + L_d + f_{pl,PTSF} L_{pl} + \left(\frac{1 + f_{pl,PTSF}}{2} \right) L_{de} \right]}{L_t}$$

Equation 15-15

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, then 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 15-16 should be used instead of Equation 15-15:

Equation 15-16

$$PTSF_{pl} = \frac{PTSF_d \left[L_u + f_{pl,PTSF} L_{pl} + f_{pl,PTSF} L'_{de} + \left(\frac{1 - f_{pl,PTSF}}{2} \right) \left(\frac{L'^{2}_{de}}{L_{de}} \right) \right]}{L_t}$$

where all terms are as previously defined.

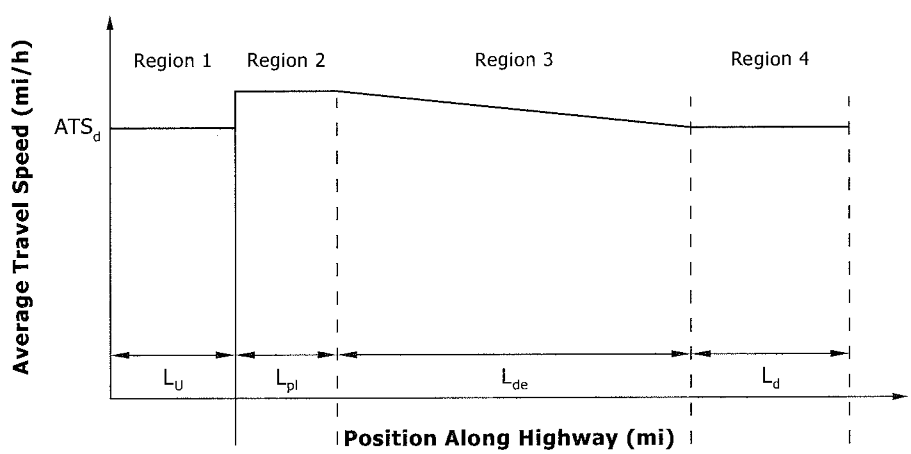
In general, the effective downstream distance of the passing lane should not be truncated. A downstream boundary short of the effective downstream distance should be considered at the point where any of the following occur:

- 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; and
- 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 15-17:

Equation 15-17

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + L_d + \left(\frac{L_{pl}}{f_{pl,ATS}} \right) + \left(\frac{2L_{de}}{1 + f_{pl,ATS}} \right)}$$

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 15-18 is used instead of Equation 15-17 to compute ATS.

Equation 15-18

$$ATS_{pl} = \frac{ATS_d L_t}{L_u + \dfrac{L_{pl}}{f_{pl,ATS}} + \dfrac{2L'_{de}}{\left[1 + f_{pl,ATS} + \left(f_{pl,ATS} - 1\right)\left(\dfrac{L_{de} - L'_{de}}{L_{de}}\right)\right]}}$$

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.

Directional Segments with Climbing Lanes on Upgrades

A climbing lane is, in effect, a passing lane added on an upgrade to allow traffic to pass heavy vehicles whose speeds are reduced. Generally, a 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 (*7*) indicates that climbing lanes on two-lane highways are warranted when

- 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 with the same procedures as passing lanes in level or rolling terrain, 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_{de} is also generally set to zero unless the climbing lane ends before the grade does. In this case, a value less than the values typically used 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

LOS Assessment for Directional Two-Lane Facilities

Two-lane highway segments have uniform characteristics that provide a basis for their analysis. Several contiguous two-lane highway segments (in the same directions) may be combined to look at a longer section (with varying characteristics) as a facility. A separate operational analysis would have to be done for each uniform segment within the facility.

Weighted-average values of PTSF and ATS may be estimated for the facility. The weighting is on the basis of total travel time within the 15-min analysis period. The total travel time of all vehicles within the 15-min analysis period is estimated with Equation 15-19 and Equation 15-20:

Equation 15-19

$$VMT_{i15} = 0.25\left(\frac{V_i}{PHF}\right)L_t$$

Equation 15-20

$$TT_{i15} = \frac{VMT_{i15}}{ATS_i}$$

where

VMT_{i15} = total vehicle miles traveled by all vehicles in directional segment i during the 15-min analysis period (veh-mi),

V_i = demand volume in directional segment i (veh/h),

PHF = peak hour factor,

L_t = total length of directional segment i (mi),

TT_{i15} = total travel time consumed by all vehicles traversing directional segment i during the 15-min analysis period (veh-h), and

ATS_i = average travel speed for directional segment i (mi/h).

Once the total travel time for all vehicles in each segment is computed, weighted-average values of PTSF and ATS can be obtained with Equation 15-21 and Equation 15-22:

$$ATS_F = \frac{VMT_1 + VMT_2 + VMT_3 + ... + VMT_i}{TT_1 + TT_2 + TT_3 + ... + TT_i}$$

Equation 15-21

$$PTSF_F = \frac{(TT_1 \times PTSF_1) + (TT_2 \times PTSF_2) + (TT_3 \times PTSF_3) + ... + (TT_i \times PTSF_i)}{TT_1 + TT_2 + TT_3 + ... + TT_i}$$

Equation 15-22

where

ATS_F = average travel speed for the facility (mi/h),

$PTSF_F$ = percent time-spent-following for the facility (decimal),

$PTSF_i$ = percent time-spent-following for segment i (decimal),

VMT_i = vehicle miles traveled for segment i (veh-mi), and

TT_i = total travel time of all vehicles in segment i (veh-h).

When a facility is put together, two-lane highway segments of different classes should not be combined. Levels of service for the facility are still based on the criteria of Exhibit 15-3. Class III two-lane highways generally only exist in short segments and would not be expected to cover a distance long enough to form a facility.

Other Performance Measures

This chapter provides detailed methodologies for estimating three measures of effectiveness that are used (depending on the highway class) to determine LOS:

- ATS (mi/h, Class I and Class III highways),
- PTSF (decimal, Class I and Class II highways), and
- PFFS (decimal, Class III highways).

In the previous section, two additional measures were introduced that can be considered as performance measures, even though they are not used to determine LOS. Equation 15-19 and Equation 15-20 can be used to estimate

- Total vehicle miles traveled by all vehicles in the analysis segment during the 15-min analysis period VMT_{i15} (veh-mi), and
- Total travel time consumed by all vehicles traversing the analysis segment during the 15-min analysis period TT_{i15} (veh-h).

These values may also be of interest in fully understanding the operational quality of the study segment.

A volume-to-capacity (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 15-23:

Equation 15-23

$$v/c = \frac{v_d}{1{,}700}$$

where v_d is the directional demand flow rate, converted to equivalent base conditions.

The difficulty in this 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.

BICYCLE MODE

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, indicating 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 (*4*). The model fits independent variables associated with roadway characteristics to the results of a 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.

Step 1: Gather Input Data

The methodology requires gathering the following input data for the facility in question:

1. Lane width (ft),
2. Shoulder width (ft),
3. Hourly directional motorized vehicle volume (veh/h),
4. Number of directional through lanes (needed for multilane highways),
5. Percentage of heavy vehicles (decimal),
6. Posted speed limit (mi/h),
7. Percentage of segment with occupied on-highway parking (decimal), and
8. Pavement rating.

Pavement rating is determined by using FHWA's 5-point present serviceability rating scale (*8*): 1 (very poor), 2 (poor), 3 (fair), 4 (good), and 5 (very good). Where data for specific variables are not available, default values may be used as shown in Exhibit 15-5.

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:

$$W_e = W_v + W_s - (\%OHP \times 10\text{ ft})$$

Equation 15-25

If W_s is greater than or equal to 4 ft and less than 8 ft:

$$W_e = W_v + W_s - 2 \times (\%OHP(2\text{ ft} + W_s))$$

Equation 15-26

If W_s is less than 4 ft:

$$W_e = W_v + (\%OHP(2\text{ ft} + W_s))$$

Equation 15-27

with, if V is greater than 160 veh/h:

$$W_v = W_{OL} + W_s$$

Equation 15-28

Otherwise,

$$W_v = (W_{OL} + W_s) \times (2 - 0.005V)$$

Equation 15-29

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 (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 instance, 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

$$BLOS = 0.507\ln(v_{OL}) + 0.1999S_t(1 + 10.38HV)^2 + 7.066(1/P)^2 - 0.005(W_e)^2 + 0.057$$

where

$BLOS$ = bicycle level of service score;

v_{OL} = directional demand flow rate in the outside lane (veh/h);

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.

3. APPLICATIONS

This chapter provides methodologies for the analysis of two-lane highway uninterrupted-flow segments that serve a wide variety of travel purposes. The procedures are most easily applied in the operational analysis mode to determine the capacity and LOS of a two-lane highway segment with known characteristics. Other applications are also possible.

DEFAULT VALUES

A detailed report on the use of default values in uninterrupted-flow analysis, including the analysis of two-lane highways, is given elsewhere (*4*). Specific default values for use with the methodology of this chapter were given in Exhibit 15-5. Default values may also be based on local estimates developed from past observations of a specific site or similar sites in a given jurisdiction.

For operational analysis and design analysis, the use of default values should be minimized whenever possible. Every default value used to replace a field-measured or other site-specific value introduces additional uncertainty into the estimation process and into the accuracy of results. Nevertheless, where no site-specific values are available, default values allow at least an approximate analysis of the situation. For planning and preliminary design analysis, use of default values is generally required, since few details are available at this stage of consideration.

TYPES OF ANALYSIS

Operational Analysis

All geometric, development, and traffic-demand characteristics are provided. The LOS that is expected to exist during the analysis period is estimated. A number of alternative performance measures may also be estimated. The methodology of this chapter is most easily used in this mode.

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 special procedures outlined in this chapter to consider the impacts of passing lanes and climbing lanes 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.

This chapter's appendix deals with some special design issues related to 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.

SERVICE FLOW RATES, SERVICE VOLUMES, AND DAILY SERVICE VOLUMES

Service flow rates, service volumes, and daily service volumes are useful concepts that can be used in the analysis of many types of facilities, including two-lane highways. The three terms must be clearly understood, because they are very different.

1. Service flow rates SF_i represent the maximum directional rate of flow that can be accommodated by a segment while maintaining the designated LOS *i*.
2. Service volumes SV_i represent the maximum directional hourly volume that can be accommodated by a segment while maintaining the designated LOS *i* during the worst 15-min period of the hour.
3. Daily service volumes DSV_i represent the maximum AADT that can be accommodated by a segment while maintaining the designated LOS *i* during the worst 15 min of the peak hour of the day, in the highest-flow direction.

In general, service flow rates and service volumes are directional values, while the daily service volume is usually stated as total traffic in both directions (since that is how AADT is stated).

The service flow rate for a particular LOS is estimated by using the methodology for the segment type under study (two-lane highways in this chapter). Equation 15-32 is then used to estimate service volume for a segment, and Equation 15-33 is used to estimate daily service volume for a segment.

$$SV_i = SF_i \times PHF$$

Equation 15-32

$$DSV_i = \frac{SV_i}{K \times D}$$

Equation 15-33

where K is the proportion of traffic occurring in the peak hour for the study segment and D is the proportion of traffic occurring in the peak direction for the study segment.

For two-lane highways, several complications arise. While all analyses of two-lane highways are for one direction, the two directions interact. Thus, if a two-way daily service volume is estimated by using the service flow rate in one direction, and then again in the other direction, different results could easily be obtained.

As with all uninterrupted-flow segments, capacity is synonymous with the service flow rate for LOS E. Thus, Equation 15-12 and Equation 15-13, presented earlier, may be used to estimate service flow rates for LOS E. Even in this case, there are two equations, since the value will depend on whether ATS or PTSF is the determining LOS parameter.

For other levels of service, the process of determining a service flow rate is more complicated. It would be beneficial if the methodology of this chapter could be used in reverse—that is, start with a value of ATS or PTSF and work backwards to the demand flow rate that would create that value. Unfortunately, virtually all of the adjustment factors used in this process depend on the demand flow rate, which is what the analyst would be trying to find. Such computations would therefore be iterative. Finding appropriate service flow rates for each LOS requires an iterative process in which different flow rates are incrementally used until the threshold for a particular LOS is found.

Once service flow rates are found, Equation 15-32 and Equation 15-33 can be used to infer service volumes and daily service volumes.

GENERALIZED DAILY SERVICE VOLUMES

Exhibit 15-30 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 15-30
Generalized Daily Service Volumes for Two-Lane Highways

The Class I—level example assumes higher speeds, with significant passing opportunities.

The Class I—rolling example assumes more moderate speeds and reduced passing opportunities because of the terrain.

The Class II—rolling example is similar to a scenic or recreational highway with lower speeds and limited passing opportunities.

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.

A number of interesting characteristics are displayed in Exhibit 15-30:

1. LOS A is not shown. Even in level terrain, it is possible to achieve this level only at very low demand flow rates (almost always lower than 50 veh/h, directional).
2. 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 capacity demand flows develop.

Exhibit 15-30 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, then 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.

USE OF ALTERNATIVE TOOLS

No alternative deterministic tools are in common use for two-lane highway analysis. Two-lane highway simulation tools are in various stages of development, but user experience with these tools is insufficient to support the formulation of useful guidance for their application to extend the scope of the procedures described in this chapter.

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.

It is possible to obtain additional performance measures 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 24, Concepts: Supplemental, has attracted increasing international interest. Some examples that illustrate potential uses of two-lane highway simulation are presented elsewhere (*9*).

4. EXAMPLE PROBLEMS

Exhibit 15-31
List of Example Problems

Problem Number	Description	Type of Analysis
1	Find the LOS of a Class I highway in rolling terrain	Operational analysis
2	Find the LOS of a Class II highway in rolling terrain	Operational analysis
3	Find the LOS of a Class III highway in level terrain	Operational analysis
4	Find the LOS of a Class I highway with a passing lane	Operational analysis
5	Find the future bicycle LOS of a two-lane highway	Planning analysis

EXAMPLE PROBLEM 1: CLASS I HIGHWAY LOS

The Facts

A segment of Class I two-lane highway has the following known characteristics:

- Demand volume = 1,600 pc/h (total in both directions)
- Directional split (during analysis period) = 50/50
- PHF = 0.95
- 50% no-passing zones in the analysis segment (both directions)
- Rolling terrain
- 14% trucks; 4% RVs
- 11-ft lane widths
- 4-ft usable shoulders
- 20 access points/mi
- 60-mi/h BFFS
- 10-mi segment length

Find the expected LOS in each direction on the two-lane highway segment as described.

Comments

The problem statement calls for finding the LOS in each direction on a segment in rolling terrain. Because the directional split is 50/50, the solution in one direction will be the same as the solution in the other direction, so only one operational analysis needs to be conducted. The result will apply equally to each direction.

Because this is a Class I highway, both ATS and PTSF must be estimated to determine the expected LOS.

Step 1: Input Data

All input data were specified above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2 and adjustment factors found in Exhibit 15-7 (for lane and shoulder width) and Exhibit 15-8 (for access points in both directions). For 11-ft lane widths and 4-ft usable shoulders, the adjustment factor for these features f_{LS} is 1.7 mi/h; for 20 access points/mi, the adjustment factor f_A is 5.0 mi/h. Then

$$FFS = BFFS - f_{LS} - f_A$$

$$FFS = 60.0 - 1.7 - 5.0 = 53.3 \text{ mi/h}$$

Step 3: Demand Adjustment for ATS

The demand volume must be adjusted to a flow rate in passenger cars per hour under equivalent base conditions. This is accomplished with Equation 15-3:

$$v_{i,ATS} = \frac{V_i}{PHF \times f_{g,ATS} \times f_{HV,ATS}}$$

Since the demand split is 50/50, both the analysis direction and opposing demand volumes are 1,600/2 = 800 veh/h.

The grade adjustment factor $f_{g,ATS}$ is selected from Exhibit 15-9 for rolling terrain. The table is entered with a demand flow rate v_{vph} in vehicles per hour, or 800/0.95 = 842 veh/h. By interpolation in Exhibit 15-9 between 800 and 900 veh/h, the factor is 0.99 to the nearest 0.01.

The passenger car equivalent for trucks and RVs is obtained from Exhibit 15-11, again for a demand flow rate of 842 veh/h. Again, by interpolation between 800 and 900 veh/h, the values obtained are $E_T = 1.4$ and $E_R = 1.1$. The heavy vehicle adjustment is then computed with Equation 15-4:

$$f_{HV,ATS} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV,ATS} = \frac{1}{1 + 0.14(1.4 - 1) + 0.04(1.1 - 1)}$$

$$f_{HV,ATS} = 0.943$$

Then

$$v_{d,ATS} = v_{o,ATS} = \frac{800}{0.95 \times 0.99 \times 0.943} = 902 \text{ pc/h}$$

Step 4: Estimate ATS

The ATS is estimated with Equation 15-6. The adjustment factor $f_{np,ATS}$ is found in Exhibit 15-15 for an FFS of 53.3 mi/h, 50% no-passing zones, and an opposing demand flow of 902 veh/h. This selection must use interpolation on all three scales. Note that interpolation is only to the nearest 0.1 for this adjustment factor. Exhibit 15-32 illustrates the interpolation.

Exhibit 15-32
Interpolation for ATS Adjustment Factor

v_o (veh/h)	Factor for FFS = 55 mi/h			Factor for FFS = 50 mi/h		
	40% NPZ	50% NPZ	60% NPZ	40% NPZ	50% NPZ	60% NPZ
800	0.7	*0.9*	1.1	0.6	*0.75*	0.9
902		**0.8**			**0.65**	
1,000	0.6	*0.7*	0.8	0.4	*0.55*	0.7

Notes: $f_{np,ATS}$ = 0.65 + (0.8 - 0.65) (3.3 / 5.0) = 0.749 = **0.7**.
NPZ = no-passing zones.

Then, Equation 15-6 gives the following:

$$ATS = FFS - 0.00776(v_d + v_o) - f_{np,ATS}$$

$$ATS = 53.3 - 0.00776(902 + 902) - 0.7$$

$$ATS = 53.3 - 14.0 - 0.7 = 38.6 \text{ mi/h}$$

Step 5: Demand Adjustment for PTSF

The adjusted demand used to estimate PTSF is found with Equation 15-7 and Equation 15-8. The grade adjustment factor is taken from Exhibit 15-16 for rolling terrain and a demand flow rate of 800/0.95 = 842 pc/h. Passenger car equivalents for trucks and RVs are taken from Exhibit 15-18. In both exhibits, the demand flow rate of 842 pc/h is interpolated between 800 pc/h and 900 pc/h to obtain the correct values. The following values are obtained:

$f_{g,PTSF}$ = 1.00
E_T = 1.0
E_R = 1.0

Then, use of Equation 15-8 gives the following:

$$f_{HV,PTSF} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

$$f_{HV,PTSF} = \frac{1}{1 + 0.14(1.0 - 1) + 0.04(1.0 - 1)} = 1.00$$

Equation 15-7 gives

$$v_{i,PTSF} = \frac{V_i}{PHF \times f_{g,PTSF} \times f_{HV.PTSF}}$$

$$v_{d,PTSF} = v_{o,PTSF} = \frac{800}{0.95 \times 1.00 \times 1.00} = 842 \text{ pc/h}$$

Step 6: Estimate PTSF

PTSF is estimated with Equation 15-9 and Equation 15-10. Exhibit 15-20 is used to obtain exponents *a* and *b* for Equation 15-10, and Exhibit 15-21 is used to obtain the no-passing-zone adjustment for Equation 15-9. All three require interpolation.

Exponents *a* and *b* are based on the opposing flow rate of 842 pc/h, which is interpolated between tabulated values of 800 pc/h and 1,000 pc/h. This is illustrated in Exhibit 15-33.

Exhibit 15-33
Interpolation for Exponents *a* and *b* for Equation 15-10

Opposing Flow Rate (pc/h)	*a*	*b*
800	-0.0045	0.833
842	**-0.0046**	**0.832**
1,000	-0.0049	0.829

Then, use of Equation 15-10 gives

$$BPTSF = 100\left[1 - \exp\left(a v_d^b\right)\right]$$

$$BPTSF = 100\left[1 - \exp\left(-0.0046 \times 842^{0.832}\right)\right]$$

$$BPTSF = 71.3\%$$

The adjustment factor for no-passing zones must also be interpolated in two variables. Exhibit 15-21 is entered with 50% no-passing zones, a 50/50 directional split of traffic, and a total two-way demand flow rate of 842 + 842 = 1,684 pc/h. The interpolation is illustrated in Exhibit 15-34.

Exhibit 15-34
Interpolation for $f_{np,PTSF}$ for Equation 15-9

Total Flow Rate (pc/h)	Adjustment Factor for 40% NPZ	Adjustment Factor for 50% NPZ	Adjustment Factor for 60% NPZ
1,400	23.8	*25.0*	26.2
1,684		16.6 + (25.0 - 16.6) (316 / 600) = **21.0**	
2,000	15.8	*16.6*	17.4

Note: NPZ = no-passing zones.

Then, use of Equation 15-9 gives

$$PTSF = BPTSF + f_{np,PTSF}\left(\frac{v_{d,PTSF}}{v_{d,PTSF} + v_{o,PTSF}}\right)$$

$$PTSF = 71.3 + 21.0\left(\frac{842}{842 + 842}\right) = 81.8\%$$

Step 7: Estimate PFFS

This step is only used for Class III highways.

Step 8: Determine LOS and Capacity

LOS is determined by comparing the estimated values of ATS and PTSF with the criteria of Exhibit 15-3. An ATS of 38.6 mi/h suggests that LOS E will exist. A PTSF of 81.8% suggests that LOS E will exist. Thus, both criteria lead to the conclusion that the segment will operate at LOS E.

Capacity is determined by either Equation 15-12 or Equation 15-13, whichever produces the lower estimate. Note, however, that all adjustment factors for use in these equations are based on a directional flow rate greater than 900 pc/h. Thus, the grade factor will be 1.00 for both ATS and PTSF. The passenger car equivalent for trucks is 1.3 for ATS and 1.00 for PTSF; the passenger car equivalent for RVs is 1.1 for ATS and 1.00 for PTSF.

The adjustment factors for heavy vehicles are as follows:

$$f_{HV,ATS} = \frac{1}{1 + 0.14\,(1.3 - 1) + 0.04\,(1.1 - 1)} = 0.96$$

$$f_{HV,PTSF} = \frac{1}{1 + 0.14\,(1.0 - 1) + 0.04\,(1.0 - 1)} = 1.00$$

and

$$c_{dATS} = 1700 \times f_{g,ATS} \times f_{HV,ATS} = 1{,}700 \times 1.00 \times 0.960 = 1{,}632 \text{ veh/h}$$

$$c_{dPTSF} = 1700 \times f_{g,PTSF} \times f_{HV,PTSF} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

Obviously, the first value holds, and the directional capacity of this facility is 1,632 veh/h. Given the 50/50 directional distribution, the two-way capacity of the segment is 1,632 + 1,632 = 3,264 veh/h. Because this exceeds the limiting capacity of 3,200 pc/h, the directional capacity cannot be achieved with a 50/50 directional distribution. A total two-way capacity of 3,200 pc/h would prevail. In terms of prevailing conditions, the capacity would be 3,200 × 1.00 × 0.960 = 3,072 veh/h. With a 50/50 directional split, this implies a directional capacity of 3,072/2 = 1,536 veh/h.

Discussion

The two-lane highway segment as described is expected to operate poorly, within LOS E. The operation is poor despite the fact that demand is only 842/1,536 = 0.55 of capacity. Both ATS and PTSF are at unacceptable levels (38.6 mi/h and 81.8%, respectively). This solution again highlights the characteristic of two-lane highways of having poor operations at relatively low *v/c* ratios. This segment should clearly be examined for potential improvements.

Given the 50/50 directional split of traffic, results for the second direction would be identical.

EXAMPLE PROBLEM 2: CLASS II HIGHWAY LOS

The Facts

A segment of Class II highway is part of a scenic and recreational route and has the following known characteristics:

- Class II highway
- 1,050 veh/h (both directions)
- 70/30 directional split
- 5% trucks; 7% RVs
- PHF = 0.85
- 10-ft lanes; 2-ft shoulders
- BFFS = 55.0 mi/h
- Rolling terrain
- 10 access points/mi
- 60% no-passing zones

Comments

Computational Steps 3 and 4, which relate to the estimation of average highway speed, will not be included. LOS for Class II highways depends solely on PTSF. The analysis will be conducted for both the 70% direction of flow and the 30% direction of flow. This is accomplished by merely reversing the analysis direction and opposing flows.

Step 1: Input Data

All input data have been summarized above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2. Adjustment factors for lane and shoulder width (Exhibit 15-7) and access points per mile (Exhibit 15-8) are used.

Exhibit 15-7 is entered with 10-ft lanes and 2-ft shoulders. The resulting adjustment is 3.7 mi/h. Exhibit 15-8 is entered with 10 access points/mi. The resulting adjustment is 2.5 mi/h. The FFS is then estimated as follows:

$$FFS = 55.0 - 3.7 - 2.5 = 48.8 \text{ mi/h}$$

Steps 3 and 4

Steps 3 and 4 are not required for Class II highways.

Step 5: Demand Adjustment for PTSF

Equation 15-7 and Equation 15-8 are used to adjust analysis direction and opposing demands to flow rates under equivalent base conditions. With a 70/30 split of traffic, the two demands are as follows:

$$V_{70\%} = V_1 = 1{,}050 \times 0.70 = 735 \text{ veh/h}$$

$$V_{30\%} = V_2 = 1{,}050 \times 0.30 = 315 \text{ veh/h}$$

In this solution, directions will be referred to as 1 and 2. Since both directions are to be analyzed, their position as "analysis direction" and "opposing" will depend on which direction is under study.

Adjustment factors both for grades (Exhibit 15-16) and for heavy vehicles (Exhibit 15-18) are needed. Exhibit 15-16 and Exhibit 15-18 are entered with a directional flow rate of 735/0.85 = 865 veh/h (Direction 1) and 315/0.85 = 371 veh/h (Direction 2). Interpolation is required in both. The following values are obtained:

$f_{g,PTSF}$ = 1.00 (Direction 1); 0.89 (Direction 2)

E_T = 1.0 (Direction 1); 1.6 (Direction 2)

E_R = 1.0 (Direction 1); 1.0 (Direction 2)

The heavy vehicle adjustment factor for both directions is computed with Equation 15-8:

$$f_{HV,PTSF1} = \frac{1}{1 + 0.05(1.00 - 1) + 0.07(1.00 - 1)} = 1.00$$

$$f_{HV,PTSF2} = \frac{1}{1+0.05(1.6-1)+0.07(1.00-1)} = 0.97$$

The adjusted demand flow rates are computed with Equation 15-7:

$$v_{1,PTSF} = \frac{735}{0.85 \times 1.00 \times 1.00} = 865 \text{ pc/h}$$

$$v_{2,PTSF} = \frac{315}{0.85 \times 0.89 \times 0.97} = 429 \text{ pc/h}$$

Step 6: Estimate PTSF

PTSF is estimated with Equation 15-9 and Equation 15-10 with values a and b taken from Exhibit 15-20 and $f_{np,PTSF}$ taken from Exhibit 15-21.

Exhibit 15-20 is entered with opposing flow rates of 429 pc/h (for Direction 1) and 865 pc/h (for Direction 2). Both values must be interpolated. The resulting values are as follows:

Direction 1: $a = -0.0024$; $b = 0.915$

Direction 2: $a = -0.0046$; $b = 0.832$

Exhibit 15-21 is entered with the total demand flow rate of 865 + 429 = 1,294 pc/h, a directional split of 70/30, and 60% no-passing zones. Interpolation is required. The factor is the same for both Directions 1 and 2:

$$f_{np,PTSF} = 23.0\%$$

BPTSF is computed with Equation 15-10:

$$BPTSF_1 = 100\left[1 - \exp(-0.0024 \times 865^{0.915})\right] = 68.9\%$$

$$BPTSF_2 = 100\left[1 - \exp(-0.0046 \times 429^{0.832})\right] = 51.0\%$$

The PTSF for each direction is computed with Equation 15-9:

$$PTSF_1 = 68.9 + 23.0\left(\frac{865}{865+429}\right) = 84.3\%$$

$$PTSF_2 = 51.0 + 23.0\left(\frac{429}{429+865}\right) = 58.6\%$$

Step 7

Step 7 is only used for Class III highways.

Step 8: Determine LOS and Capacity

The LOS is determined by comparing the PTSF values obtained with the criteria of Exhibit 15-3. Applying these criteria reveals that Direction 1 operates at LOS D, while Direction 2 operates at LOS C.

By using the adjustment selected for ≥900 veh/h, capacity is computed with Equation 15-13:

$$c_{1,PTSF} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

$$c_{2,PTSF} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

Discussion

The LOS is, at best, somewhat marginal on this two-lane highway segment, based solely on the PTSF.

The value of capacity must be carefully considered. If the directional capacities were expanded to two-way capacities based on the given demand split, the capacity in the 30% direction would imply a two-way capacity well in excess of the 3,200 pc/h limitation for both directions. Therefore, even though a capacity of 1,700 veh/h is possible in the 30% direction, it could not occur with a 70/30 demand split. In this case, the two-way capacity would be limited by the capacity in the 70% direction and would be 1,700/0.70 = 2,429 veh/h. The practical capacity for the 30% direction of flow is actually best estimated as 2,429 – 1,700 or 729 veh/h. Given that the 70/30 directional split holds, when the 30% direction reaches a demand flow rate of 729 veh/h, the opposing direction (the 70% side) would be at its capacity.

EXAMPLE PROBLEM 3: CLASS III HIGHWAY LOS

The Facts

A Class III two-lane highway runs through a rural community in level terrain. It has the following known characteristics:

- Class III highway
- Demand volume = 900 veh/h (both directions)
- 10% trucks; no RVs
- Measured FFS = 40 mi/h
- 12-ft lanes; 6-ft shoulders
- PHF = 0.88
- 80% no-passing zones
- 60/40 directional split
- 40 access points/mi
- Level terrain

Comments

Because this is a Class III highway, LOS will be based on PFFS. Thus, Steps 5 and 6, which relate to the estimation of PTSF, will not be used.

Step 1: Input Data

All input data are specified above.

Step 2: Estimate FFS

A measured FFS is specified: 40 mi/h.

Step 3: Demand Adjustment for ATS

The total demand volume of 900 veh/h must be separated into two directional flows. Since both directions will be evaluated, directions are labeled 1 and 2.

$$V_1 = 900 \times 0.60 = 540 \text{ veh/h}$$

$$V_2 = 900 \times 0.40 = 360 \text{ veh/h}$$

The adjusted demand flow rate in passenger cars per hour under equivalent base conditions is estimated with Equation 15-3. A grade adjustment factor is selected from Exhibit 15-9, and passenger car equivalents for trucks are selected from Exhibit 15-11. Both exhibits are entered with a demand flow rate in vehicles per hour:

$$v_1 = 540 / 0.88 = 614 \text{ veh/h}$$

$$v_2 = 360 / 0.88 = 409 \text{ veh/h}$$

The following values are selected from Exhibit 15-9 and Exhibit 15-11. In all cases, interpolation is required:

Value	Direction 1	Direction 2
$f_{g,ATS}$	1.00	1.00
E_T	1.1	1.3

Then, use of Equation 15-4 gives

$$f_{HV,ATS(1)} = \frac{1}{1 + 0.10(1.1 - 1)} = 0.99$$

$$f_{HV,ATS(2)} = \frac{1}{1 + 0.10(1.3 - 1)} = 0.97$$

Use of Equation 15-3 gives

$$v_{1ATS} = \frac{540}{0.88 \times 1.00 \times 0.99} = 620 \text{ pc/h}$$

$$v_{2ATS} = \frac{360}{0.88 \times 1.00 \times 0.97} = 422 \text{ pc/h}$$

Step 4: Estimate ATS

ATS is estimated with Equation 15-6 with an adjustment factor for no-passing zones taken from Exhibit 15-15. The adjustment factor is based on a 40-mi/h FFS and 80% no-passing zones. Interpolating for an opposing demand flow rate of 422 pc/h (Direction 1) and 620 pc/h (Direction 2) gives the following:

$$f_{np,ATS(1)} = 2.4 \text{ mi/h}$$

$$f_{np,ATS(2)} = 1.6 \text{ mi/h}$$

Then, use of Equation 15-6 gives

$$ATS_1 = 40.0 - 0.00776(620 + 422) - 2.4 = 29.5 \text{ mi/h}$$

$$ATS_2 = 40.0 - 0.00776(422 + 620) - 1.6 = 30.3 \text{ mi/h}$$

Steps 5 and 6

Steps 5 and 6 are not used for Class III highways.

Step 7: Estimate PFFS

The LOS for Class III facilities is based on PFFS achieved, or ATS/FFS. For this segment PFFS is as follows:

$$PFFS_1 = 29.5 / 40.0 = 73.8\%$$

$$PFFS_2 = 30.3 / 40.0 = 75.8\%$$

Step 8: Determine LOS and Capacity

From Exhibit 15-3, the LOS for Direction 1 is D, while the LOS for Direction 2 is C. The two values of PFFS are close, but the boundary condition between LOS C and D is 0.75. To be LOS C, PFFS must exceed 0.75, and it is just below the threshold in Direction 1 and just above the threshold in Direction 2.

Capacity is evaluated with adjustment factors for ≥900 pc/h in level terrain. This makes all adjustment factors 1.00 (for ATS). Thus, the capacity in either direction is as follows:

$$c_{1,ATS} = c_{2,ATS} = 1{,}700 \times 1.00 \times 1.00 = 1{,}700 \text{ veh/h}$$

The two-way capacity values implied are 1,700/0.60 = 2,833 veh/h (Direction 1) and 1,700/0.40 = 4,250 veh/h (Direction 2). Obviously, the implied two-way capacity is the 2,833 veh/h. Moreover, this suggests that the directional capacity in Direction 2 cannot be achieved with a 60/40 demand split. Rather, the directional capacity in Direction 2 occurs when the capacity in Direction 1 occurs, or 2,833 × 0.40 = 1,133 veh/h.

Discussion

This segment of Class III two-lane highway operates just at the LOS C–D boundary. Depending on the length of the segment and local expectations, this may or may not be acceptable.

EXAMPLE PROBLEM 4: CLASS I HIGHWAY LOS WITH A PASSING LANE

The Facts

The 10-mi segment of the two-lane highway analyzed in Example Problem 1 will be improved with 2-mi passing lanes (one in each direction), both installed at 1.00 mi from the segment's beginning. The segment without a passing lane has already been analyzed, and the results of that analysis are listed below:

- Demand volume = 800 veh/h in each direction
- Demand flow rate (ATS) = 902 pc/h in each direction
- Demand flow rate (PTSF) = 842 pc/h in each directions

- FFS = 53.3 mi/h
- ATS = 38.6 mi/h
- PTSF = 81.8%
- Rolling terrain
- PHF = 0.95

Comments

Both directions will involve the same computations, since the directional distribution is 50/50, and in both cases, the passing lane will start 1.00 mi after the beginning of the segment (of 10 mi) and will end 3.00 mi after the beginning of the segment.

Step 1: Conduct an Analysis Without the Passing Lane

Completed as Example Problem 1.

Step 2: Divide the Segment into Regions

Exhibit 15-35 shows the division of the 6-mi segment into regions. The effective downstream length of the passing lane is selected from Exhibit 15-23 (value is different for ATS and PTSF) for a demand flow rate of 800/0.95 = 842 veh/h.

Exhibit 15-35
Region Lengths for Use in Example Problem 4

To Determine	L_u (mi)	L_{pl} (mi)	L_{de} (mi) Exhibit 15-23	L_d (mi) Equation 15-14
ATS	1.00	2.00	1.7	5.3
PTSF	1.00	2.00	4.7	2.3

Step 3: Determine the PTSF

The PTSF, as affected by the presence of a passing lane, is estimated with Equation 15-15 and an adjustment factor selected from Exhibit 15-26. The adjustment factor $f_{pl,PTSF}$ is 0.62. Then

$$PTSF_{pl} = \frac{PTSF\left[L_u + L_d + f_{pl,PTSF}L_{pl} + \left(\frac{1+f_{pl,PTSF}}{2}\right)L_{de}\right]}{L_t}$$

$$PTSF_{pl} = \frac{81.8\left[1.0 + 2.3 + (0.62 \times 2.00) + \left(\frac{1+0.62}{2}\right)4.7\right]}{10}$$

$$PTSF_{pl} = \frac{81.8(3.3 + 1.24 + 3.81)}{10} = \frac{81.8 \times 8.35}{10} = 68.3\%$$

Step 4: Determine the ATS

The ATS as affected by the presence of a passing lane is found with Equation 15-17 and an adjustment factor selected from Exhibit 15-28. The adjustment factor selected is 1.11. Then

$$ATS_{pl} = \frac{ATSL_t}{L_u + L_d + \left(\frac{L_{pl}}{f_{pl,ATS}}\right) + \left(\frac{2L_{de}}{1 + f_{pl,ATS}}\right)}$$

$$ATS_{pl} = \frac{38.6 \times 10}{1.00 + 5.3 + \left(\frac{2.00}{1.11}\right) + \left(\frac{2 \times 11.7}{1 + 1.11}\right)}$$

$$ATS_{pl} = \frac{38.6 \times 110}{6.30 + 1.80 + 1.61} = \frac{38.6 \times 110}{9.71} = 39.7 \text{ mi/h}$$

Step 5: Determine the LOS

Exhibit 15-3 shows that the LOS, as determined by PTSF, has improved to D. The LOS determined by ATS remains E. Thus, while PTSF has improved significantly, the ATS has not improved enough to improve the overall LOS, which remains E.

Discussion

Adding a 2-mi passing lane to a 10-mi segment of Class I highway operating at LOS E was insufficient to improve the overall LOS, although the PTSF did improve from 81.8% to 68.3%. It is likely that a longer (or a second) passing lane would be needed to improve the ATS sufficiently to result in LOS C or LOS D.

EXAMPLE PROBLEM 5: TWO-LANE HIGHWAY BICYCLE LOS

A segment of two-lane highway (without passing lanes) is being evaluated for potential widening, realigning, and repaving. Analyze the impacts of the proposed project on the bicycle LOS in the peak direction.

The Facts

The roadway currently has the following characteristics:

- Lane width = 12 ft
- Shoulder width = 2 ft
- Pavement rating = 3 (fair)
- Posted speed limit = 50 mi/h
- Hourly directional volume = 500 veh/h (no growth is expected)
- Percentage of heavy vehicles = 5%
- PHF = 0.90
- No on-highway parking

The proposed roadway design has the following characteristics:

- Lane width = 12 ft
- Shoulder width = 6 ft
- Pavement rating = 5 (very good)
- Posted speed limit = 55 mi/h
- No on-highway parking

Step 1: Gather Input Data

All data needed to perform the analysis are listed above.

Step 2: Calculate the Directional Flow Rate in the Outside Lane

By using the hourly directional volume and the PHF, calculate the directional demand flow rate with Equation 15-24. Because this is a two-lane highway segment without a passing lane, the number of directional lanes N is 1. Because traffic volumes are not expected to grow over the period of the analysis, v_{OL} is the same for both current and future conditions.

$$v_{OL} = \frac{V}{PHF \times N}$$

$$v_{OL} = \frac{500}{0.90 \times 1} = 556 \text{ veh/h}$$

Step 3: Calculate the Effective Width

For current conditions, the hourly directional demand V is greater than 160 veh/h and the paved shoulder width is 2 ft; therefore, Equation 15-27 and Equation 15-28 are used to determine the effective width of the outside lane. Under future conditions, the paved shoulder width will increase to 6 ft; therefore, Equation 15-26 and Equation 15-28 are used.

For current conditions:

$$W_v = W_{OL} + W_s = 12 \text{ ft} + 2 \text{ ft} = 14 \text{ ft}$$

$$W_e = W_v + (\%OHP(2 \text{ ft} + W_s))$$

$$W_e = 14 \text{ ft} + (0 \times (2 \text{ ft} + 2 \text{ ft})) = 14 \text{ ft}$$

Under the proposed design:

$$W_v = W_{OL} + W_s = 12 \text{ ft} + 6 \text{ ft} = 18 \text{ ft}$$

$$W_e = W_v + W_s - 2 \times (\%OHP(2 \text{ ft} + W_s))$$

$$W_e = 18 \text{ ft} + 6 \text{ ft} - 2 \times (0 \times (2 \text{ ft} + 6 \text{ ft})) = 24 \text{ ft}$$

Step 4: Calculate the Effective Speed Factor

Equation 15-30 is used to calculate the effective speed factor. Under current conditions:

$$S_t = 1.1199 \ln(S_p - 20) + 0.8103$$

$$S_t = 1.1199\ln(50-20)+0.8103 = 4.62$$

Under the proposed design:

$$S_t = 1.1199\ln(55-20)+0.8103 = 4.79$$

Step 5: Determine the LOS

Equation 15-31 is used to calculate the bicycle LOS score, which is then used in Exhibit 15-4 to determine the LOS. Under existing conditions:

$$BLOS = 0.507\ln(v_{OL}) + 0.1999 S_t(1+10.38HV)^2 + 7.066(1/P)^2 - 0.005(W_e)^2 + 0.057$$

$$BLOS = 0.507\ln(556) + 0.1999(4.62)(1+10.38\times 0.05)^2 + 7.066(1/3)^2 - 0.005(14)^2 + 0.057$$

$$BLOS = 3.205 + 2.131 + 0.785 - 0.980 + 0.057 = 5.20$$

Therefore, the bicycle LOS for existing conditions is LOS E. Use of the same process for the proposed design results in the following:

$$BLOS = 0.507\ln(556) + 0.1999(4.79)(1+10.38\times 0.05)^2 + 7.066(1/5)^2 - 0.005(24)^2 + 0.057$$

$$BLOS = 3.205 + 2.209 + 0.283 - 2.880 + 0.057 = 2.87$$

The corresponding LOS for the proposed design is LOS C.

Discussion

Although the posted speed would increase as a result of the proposed design, this negative impact on bicyclists would be more than offset by the proposed shoulder widening, as indicated by the improvement from LOS E to LOS C.

5. REFERENCES

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

1. Harwood, D. W., A. D. May, Jr., I. B. Anderson, and A. R. Archilla. *Capacity and Quality of Service of Two-Lane Highways*. Final report, NCHRP Project 3-55(3). Midwest Research Institute, Kansas City, Mo., 1999.
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7. *A Policy on Geometric Design of Streets and Highways*. American Association of State Highway and Transportation Officials, Washington, D.C., 2004.
8. *Highway Performance Monitoring System Field Manual*, Chapter 4. Federal Highway Administration, Washington, D.C., May 2005.
9. Courage, K. G., S. Washburn, L. Elefteriadou, and D. Nam. *Guidance for the Use of Alternative Traffic Analysis Tools in Highway Capacity Analyses*. National Cooperative Highway Research Program Project 3-85 Final Report. University of Florida, Gainesville, 2010.

APPENDIX A: 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. Some two-lane highways, however, 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 are included in this chapter.

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.

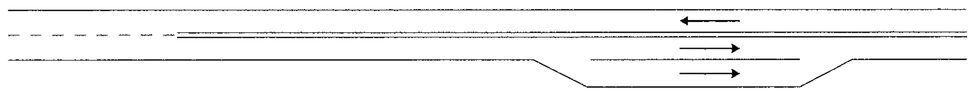
No calculation methodologies are provided in this chapter for these treatments; however, the treatments are discussed below to indicate their potential for improving traffic operations on two-lane highways.

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 vehicle 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, however, 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-A1 illustrates a typical turnout.

Exhibit 15-A1
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 (*A1, A2*). Turnouts have been found to operate safely, with experts (*A2–A4*) 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 then returns to the travel lane once the passing vehicles 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 (*A1, A2*) 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.

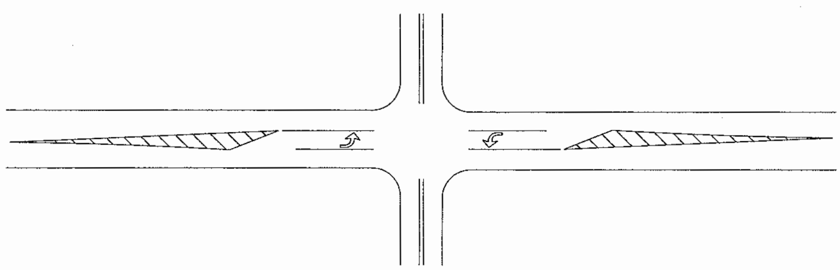
Research has shown that speeds at low traffic volumes tend to increase on wider lanes, but the effect on speeds at higher volumes varies (*A5*). More than 70% of drivers indicated that they appreciate the increased passing opportunities available on the wider lanes. No safety problems have been associated 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. It is reasonable to estimate the traffic operational performance on a directional two-lane highway segment containing wider lanes as midway between the segment with and without a passing lane of comparable length.

INTERSECTION TURN LANES

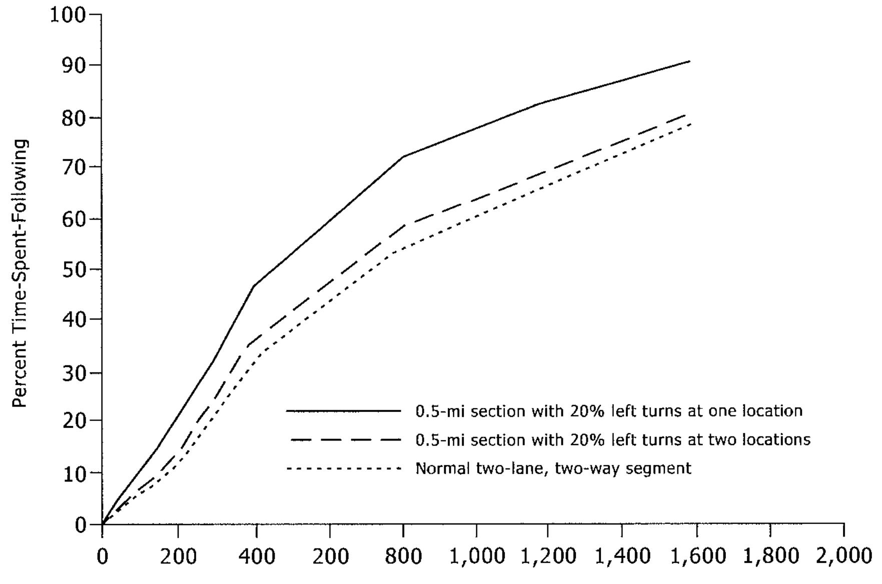
Intersection turn lanes are desirable at selected locations on two-lane highways to reduce delays to through vehicles cased by turning vehicles and to reduce turning accidents. Separate right- and left-turn lanes may be provided, as appropriate, to remove turning vehicles from the through travel lanes. 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-A2 shows a typical two-lane highway with left-turn lanes at an intersection.

Exhibit 15-A2
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 (*A6*). The HCM's intersection analysis methodologies can be used to quantify the effects of intersection turn lanes on signalized and unsignalized intersections. There is no methodology, however, 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 (*A7*); the results are shown in Exhibit 15-A3. 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 has the potential to minimize these effects.

Exhibit 15-A3
Effect of Turning Delays at Intersections on PTSF

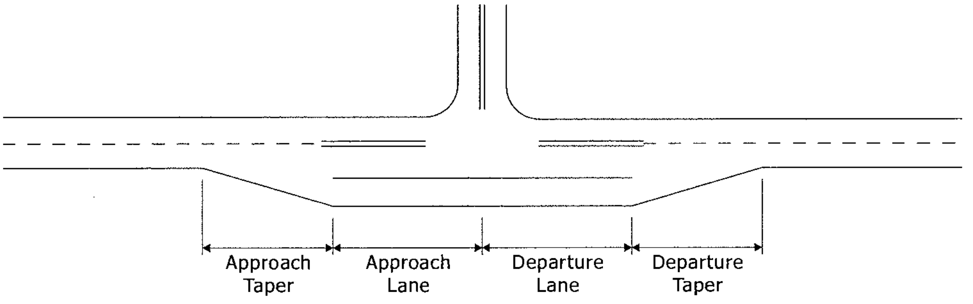


Source: Hoban (*A7*).

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-A4, 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 (*A8*).

Exhibit 15-A4
Typical Shoulder Bypass Lane at a Three-Leg Intersection on a Two-Lane Highway



TWO-WAY LEFT-TURN LANES

A two-way left-turn lane (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-A5 illustrates a typical TWLTL.

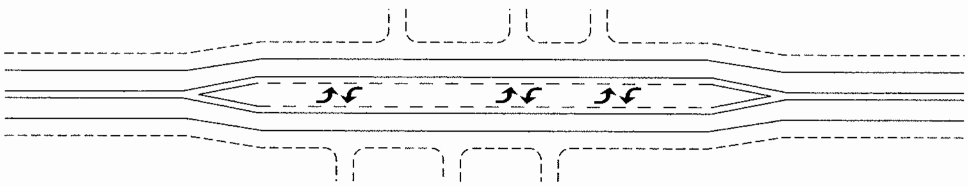


Exhibit 15-A5
Typical TWLTL on a Two-Lane Highway

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 (*A2*). 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 (*A2*). 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-A6 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.

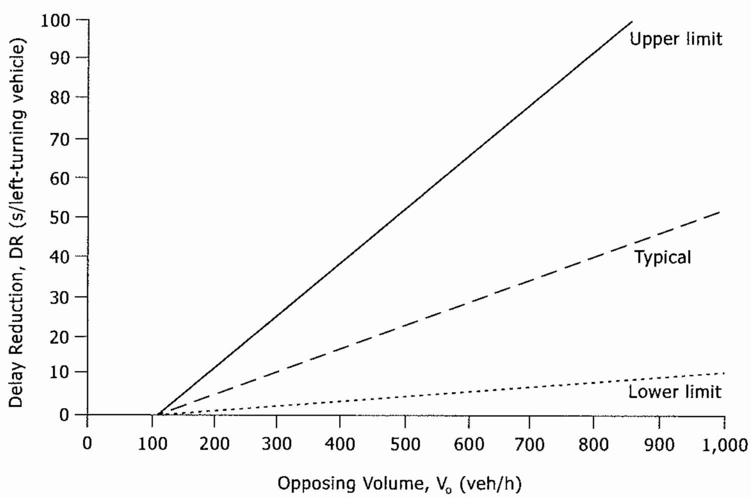


Exhibit 15-A6
Estimated Delay Reduction with a TWLTL on a Two-Lane Highway Without Paved Shoulders

Source: Harwood and St. John (*A2*).

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

REFERENCES

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A2. Harwood, D. W., and A. D. St. John. *Passing Lanes and Other Operational Improvements on Two-Lane Highways*. Report FHWA/RD-85/028. U.S. Department of Transportation, Washington, D.C., July 1984.

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A7. Hoban, C. J. *A Model for Predicting Turning and Platooning Delays.* Internal Report AIR 359-16. Australian Road Research Board, 1986.

A8. Sebastian, O. L., and R. S. Pusey. *Paved-Shoulder Left-Turn By-Pass Lanes: A Report on the Delaware Experience.* Delaware Department of Transportation, Oct. 1982.

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.

A

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