

11 SEGMENT AND FACILITY ANALYSIS

11.1 Purpose

Freeways and multilane highways serve more than half the vehicle miles travelled in Oregon (FHWA 2016). In urban areas, they serve as major commute routes and provide access to intermodal facilities, while in rural areas, they facilitate both intrastate and interstate movement of persons and freight. Recognizing their importance to Oregon's economy and quality of life, and the significant investment made in developing these facilities, the Oregon Highway Plan sets more restrictive mobility targets (i.e., lower volume-to-capacity [v/c] ratios) for these facilities than for other classes of roadways.

This chapter also provides procedures for the analysis of rural two-lane highways, which make up the largest percentage of the state highway mileage. These highways cover a wide range of geographical and topographical conditions and connect all parts of the state.

To assure that mobility targets and other key indicators of mobility are explicitly integrated into freeway and highway analyses in the state of Oregon, this chapter provides a range of procedures that are scaled to reflect analysis complexity, regional context, and study scope. These methods are anchored in national policy and guidance documents, including the *Highway Capacity Manual* (HCM) 7th Edition and its companion *Planning and Preliminary Engineering Applications Guide* (PPEAG). However, the methods have been customized through the incorporation of Oregon-specific default values and best practices that reflect the diverse nature of Oregon's freeway and highway operations, including differences between urban and rural facilities. The guidance in this chapter considers project context, project type, data availability, and level of effort needed to conduct an evaluation. The goal of the guidance is to balance resource needs and complexity with desired analysis outcomes and performance measures.

This chapter's methods can be used to evaluate the operations of freeways and the uninterrupted-flow portions of multilane and two-lane highways (i.e., roadway sections without traffic signals, roundabouts, or other forms of intersection control requiring highway traffic to potentially stop or yield). Two general categories of methods are provided: those applying to roadway *segments* and those applying to roadway *facilities*. Segments are sections of roadway with similar traffic demands and geometric characteristics, while facilities are composed of multiple contiguous segments.

Segment analysis is typically used to evaluate the v/c ratio of a given roadway section and as a prerequisite for performing a facility analysis. A facility analysis is typically used to evaluate other kinds of performance measures, such as travel time, travel speed, vehicle hours of delay, and measures of congestion, and to evaluate roadway operations when demand exceeds capacity, including identifying bottlenecks and the extent of queues. A facility analysis is also used for more specialized types of analyses involving work zones, managed lanes, travel time reliability, and active traffic management strategies.

To support planning for a future in which some of the motor vehicles on freeways are no longer human-driven, but instead are connected and automated vehicles (CAVs), this chapter includes guidance on adjusting the future capacity of freeway segments and facilities in planning scenarios where CAVs are assumed to be part of the traffic stream. This guidance is based on the capacity adjustment factors (CAFs) for freeway segments presented in the HCM, which were developed by a multi-state pooled-fund study led by ODOT. No guidance is presented for the effects of CAVs on multilane or two-lane highway capacity because no research has been conducted yet for these roadway types.



As of 2022, no vehicles were available commercially that met the definition of a CAV for the purposes of freeway analysis (i.e., a vehicle with an operating cooperative adaptive cruise control system that is capable of communicating with other vehicles and driving without human intervention in any freeway situation). The CAFs for CAVs are intended for use only in longer-range planning analyses at the broad brush or screening analysis levels (defined below). See Appendix 6B for more information about CAVs, including guidance on estimating the percentage of CAVs in the traffic stream in a future year, tables of CAF values, and example problems.

11.2 Overview of Analysis Levels, Applications, Methods, and Tools

11.2.1 Introduction

This section provides guidance on matching the appropriate level of analysis detail to the type of highway analysis being performed, along with guidance on the available tools that support different levels of analysis. Using a data-intensive analysis method for large-scale, long-range analyses may be an inefficient use of resources and may produce results that imply greater precision than is possible given the limitations of the available data. At the same time, using a method that is too generalized will not provide the detail needed to adequately evaluate alternatives and reach final decisions. Therefore, this section describes a range of methods that can be applied as a project progresses from identifying needs to developing, analyzing, and prioritizing alternatives, and finally to project development. It is recognized that the methods described in this section may not be applicable to every analysis situation that may be encountered; therefore, guidance is also provided for when alternative analysis tools, including traffic simulation, may be appropriate to supplement the analytical approaches presented in Chapter 11.

11.2.2 Analysis Levels

Section 2.3.2 introduced the concept of analysis levels. The analysis level used in a particular study depends on several factors, including the size of the study area, the amount of data already available or to be collected, the assumptions used to develop the input data (e.g., measured volumes versus 20-year traffic forecasts), the desired performance measures, and the intended use of the analysis results for decision making. The three levels applicable to freeway and multilane highway analysis are:

- **Broad brush**, a high-level roadway capacity analysis requiring minimal amounts of data and incorporating several assumptions or default values. The HCM and PPEAG refer to this analysis level as a *planning analysis* using *service volume tables*.
- **Screening**, a medium-level analysis applicable when more input data are available, but no final decisions about roadway design elements, traffic control, or project approval will be made because of the analysis. As the name implies, this level is often used to screen and prioritize several alternatives as part of a long-range planning effort. The PPEAG refers to this analysis level as a *planning analysis* using *simplified HCM methods* and/or the *HCM method with default values*.
- **Detailed**, a low-level analysis in which all or nearly all input data are known, and the analysis results will be used to make final decisions about roadway design elements, traffic control, and/or project approval. This type of analysis considers the widest number of factors that can influence roadway performance and generates the largest number of potential performance measures. The HCM and PPEAG refer to this analysis level as an *operations analysis* when the analysis is used to determine roadway performance. Other examples of detailed analyses include *design analysis*, where the HCM operations method is applied iteratively to determine the roadway geometry that achieves a desired roadway performance, and *analysis using detailed alternative tools*, such as simulation.

Exhibit 11-1 compares the ODOT and HCM/PPEAG analysis levels.

Exhibit 11-1 ODOT and HCM Analysis Levels Compared

ODOT Analysis Level	HCM/PPEAG Analysis Level	HCM/PPEAG Analysis Method
Broad Brush	Planning	Service volume tables
Screening	Planning	PPEAG: Simplified HCM method HCM/PPEAG: Operations method using default values
Detailed	Operations	HCM operations method (roadway performance, given known roadway/traffic characteristics)
	Design	HCM operations method (roadway geometry, given desired roadway performance)
	Alternative Tool	Simulation or other detailed alternative tool

11.2.3 Analysis Methods

This section summarizes the methods for highway analysis that are presented in detail in Section 11.3. The methods vary in their level of detail, the number of performance measures produced, and the roadway length analyzed. Alternative analysis tools, including simulation and tools that draw from national and private vendor databases, are also discussed briefly, because they can be used to address limitations of HCM-based analysis methods, to calibrate HCM methods, and/or to generate additional performance measures, among other potential uses.

Broad Brush

Tables providing generalized roadway capacity values are a broad-brush tool that can be used to quickly estimate a v/c ratio, requiring only knowledge of daily or peak-hour volumes, the number of directional lanes on the roadway, the generalized terrain (level, rolling, or mountainous), and the area type (urban or rural). These tables are developed directly from the detailed HCM method for determining capacity and apply default values for all required inputs not directly specified in the table. Thus, the accuracy of the results obtained from using such a table depends on how well the roadway's geometric and traffic characteristics match the assumptions used in developing the table. Results can be improved when more information is known about the facility—for example, the heavy vehicle percentage—as adjustment factors can be developed to reflect differences between known roadway characteristics and the assumed characteristics used to develop the table; this approach to using generalized capacity tables is taken in this chapter. Generalized capacity tables are provided for freeway and multilane highway facilities.

Screening

The screening methods presented in this chapter are simplified versions of detailed HCM methods, as presented in the PPEAG. Depending on the complexity of a given HCM method, the simplifications range from recommended default values to apply with the HCM method to separate methods that incorporate only the most important variables that influence the performance measure results. Screening methods are provided for all freeway and multilane highway segment and facility types.

Detailed

The detailed methods presented in this chapter are the methods given in the HCM 7th Edition. Detailed methods are provided for all freeway and multilane highway segment and facility types.

Alternative Tools

Alternative tools provide different approaches to estimating roadway performance than what is used in the HCM and PPEAG. They will be not discussed further in this chapter, which focuses on HCM-based methods, but the analyst should nevertheless be aware of their potential uses, including to address the limitations of HCM methods. Examples of alternative tools and their potential uses include:

- Microsimulation.** Microsimulation models the behavior of individual vehicles in a traffic stream with the resolution of very short time intervals, as opposed to HCM-based methods, which model the aggregate behavior of the vehicles in the traffic stream over 15-minute periods. With appropriate calibration, microsimulation can also be used to evaluate conditions that are beyond the capability of HCM methods, such as complex interactions with a mix of driver behavior and vehicle types, or alternative performance measures. As a result, microsimulation can be a useful tool for addressing the limitations of HCM methods, for confirming the results of an HCM analysis, and for creating visualizations of roadway operations. At the same time, because of the effort required to set up and calibrate a simulation model, microsimulation frequently is not a cost-effective tool for planning and screening applications where many different facilities and/or alternatives must be analyzed. In addition, there are some applications that HCM-based methods can perform better than simulation, including estimating capacity for a given condition (simulation uses capacity as an input) and evaluating travel time reliability. Chapters 8 and 15 provide more information about mesoscopic and microscopic simulation.
- Probe data analysis tools.** The range of tools available to evaluate existing or historical travel times/speeds and travel time reliability from commercial sources of probe data is described in Section 18.1.6. In general, each source of probe data has its own associated software tool for generating performance measures from archived travel time data. In addition, some tools exist that can process raw data obtained from any of the commercial sources to generate statistics and visualizations. These tools provide direct measurements of travel speeds and reliability (both for individual time periods and as averages), as opposed to the estimates produced by this chapter's methods. At the same time, the tools are incapable of directly measuring capacity (as only a sample of the overall traffic volume is used as probes), nor can they be used to forecast future performance. These tools work well in conjunction with HCM methods, providing existing condition information that can be used to calibrate HCM methods to local conditions, thus providing better future forecasts (see Appendix 11B for guidance on calibrating HCM methods). The use of probe data for calibration is particularly important for freeway facility and freeway reliability analyses.

11.2.4 Analysis Applications

Exhibit 11-2 summarizes the potential applicability of PPEAG methods, HCM methods, and simulation to five types of planning and engineering applications: regional transportation plans, transportation system plans (TSPs), corridor plans, refinement plans, and project development. These applications were first introduced in Section 2.3.1 and are described in more detail below.

Exhibit 11-2 Method Applicability to Common Transportation Planning and Engineering Applications

Analysis Level	Broad Brush	Screening		Detailed	
Analysis Tool Application	Generalized Capacity Tables	PPEAG Simplified Segment	PPEAG Simplified Facility	HCM Operations	Simulation
Regional Transportation Plan	○	◐	●		
Transportation System Plan	○	◐	●		
Corridor Plan	○	◐	●		
Refinement Plan				◐ or ●	●
Project Development				◐	●

Note: ○ = problem identification, ◐ = evaluation, ● = detailed evaluation.

Regional transportation plans identify the long-term (20-year or longer horizon) transportation needs of metropolitan areas exceeding 50,000 population, develop potential projects or actions to address those needs, and prioritize recommended projects and actions in the process of developing a financially constrained plan. These areas typically have at least one (and often multiple) freeways and/or multilane highways within their planning area. Generalized capacity tables are useful for an approximate planning-level estimate of the capacity of these roadways; the future demand volumes generated by the regional travel demand model can be compared to these capacities to determine whether future problems may exist. The PPEAG's simplified segment methods can be used to evaluate identified problem areas, while the PPEAG's simplified facility method can be used to preliminarily evaluate alternatives and to generate a broader range of performance measures than v/c ratios.

Transportation System Plans (TSPs) serve as the long-range transportation plan for a city or county, many of which do not have any freeways or multilane highways. Most cities that have a freeway within their planning area will only have one or two interchanges to study, but a county-wide TSP may have many miles of freeway or multilane highway to study. Like regional transportation plans, generalized capacity tables can be used to identify potential problems, the PPEAG simplified segment methods can be used to evaluate specific problem areas, and the PPEAG simplified facility method can be used for preliminary evaluation of projects being considered for the plan.

A **corridor plan** covers an entire highway (e.g., US 101) or a long-defined section of a route (e.g., Madras–Portland) to develop specific project recommendations to be implemented in the mid- to long-term. Like the previous two types of plans, generalized capacity tables can be used to identify potential problems, the PPEAG simplified segment methods can be used to evaluate specific problem areas, and the PPEAG simplified facility method can be used for preliminary evaluation of projects being considered for the plan.

A **refinement plan** studies a subarea in detail. The need for such a plan may have been identified through a TSP, where resource constraints did not permit the more-detailed analysis required to develop appropriate recommendations. Such a plan might include a single freeway interchange or a relatively short section of multilane highway but would be unlikely to study extended lengths of these roadway types (otherwise, a corridor plan would have been called for). HCM operations methods are appropriate for studying the interchange or multilane highway segment. Simulation can be considered to confirm draft recommendations, to address limitations of the HCM method, or to develop visualizations of key recommendations. But for many refinement plans, the HCM method may be sufficient even for a detailed evaluation.

At the **project development** stage, final decisions about roadway design features and traffic control are being made and planning methods are therefore inappropriate. HCM operations methods are appropriate for evaluating the project, with simulation used to confirm the results, develop visualizations, and/or address limitations of the HCM method.

11.2.5 Analysis Tools

Although a basic capacity analysis can be quickly performed by hand for an individual segment, analysis tools help speed up the analysis process, ensure accurate calculations (assuming accurate inputs), and generate additional useful performance measures. This section introduces the tools available (as of mid-2018) that implement this chapter's analysis methods, and compares the tools' capabilities, inputs and outputs, and limitations.

Tool Descriptions

The **PPEAG freeway planning tool** is a research-grade Excel spreadsheet that automates the PPEAG's freeway segment and facility methods. The spreadsheet was developed by NCHRP Project 07-22 and can be downloaded from the Applications Guides section of the HCM Volume 4 website (www.hcmvolume4.org, requires a free, one-time registration).

Highway Capacity Software (HCS) is commercial software for the Windows operating system that is developed, distributed, and supported by the McTrans Center at the University of Florida. HCS includes modules that implement the HCM's procedures for basic freeway and multilane highway segments, two-lane highways, freeway weaving segments, freeway merge and diverge segments, freeway facilities, and freeway and arterial travel time reliability.

FREEway EVALuation (FREEVAL) is open-source software developed by the Institute for Transportation Research and Education (ITRE) at North Carolina State University. It uses the Java programming language and therefore runs on both Windows and Mac computers. It implements the HCM freeway facilities method, which incorporates the methods for basic freeway segments, weaving segments, and merge and diverge segments. The tool also provides modules for the HCM's freeway travel time reliability

and active traffic and demand management (ATDM) methods. The software and its users guide can be downloaded from the Technical Reference Library (Chapter 11) section of the HCM Volume 4 website (www.hcmvolume4.org, requires a free, one-time registration). **FREEVAL-OR** is a customized version of the tool that incorporates all the Oregon-specific default values identified in the APM. For more information on the use of FREEVAL-OR refer to Appendix 11E. Note: Java updates many introduce errors but the tool is still usable.

HCM CALC is software for the Windows operating system that was developed by SwashWare and the University of Florida Research Foundation. HCM CALC includes modules that implement the HCM's procedures for basic freeway and multilane highway segments, two-lane highways, freeway weaving segments, freeway merge and diverge segments, freeway facilities, and freeway travel time reliability. It is available for free through the Microsoft store for Windows 10 users.

Tool Comparison

While the tools described above automate some or all of this chapter's methodologies, they have different interfaces, outputs, and features. The differences between the tools are outlined in the tables below. Exhibit 11-3 compares the tools' general characteristics, including:

- Installation considerations such as cost, technical requirements, and level of national recognition.
- Staffing needs including level of training required, general complexity of the tool, and available resources and support; and
- User experience including the ease of entering inputs and exporting outputs.

Exhibit 11-3 Software Tool Overview, Resource Needs, and User Experience

Overview	PPEAG Tool	HCS	FREEVAL	HCM CALC
<i>Tool Overview</i>				
Source	hcmvolume4.org	McTrans	hcmvolume4.org	University of Florida
Cost	Free	License Fee	Free	Free
Operating system	Windows/Mac	Windows	Windows/Mac	Windows 10
Installation required	No (need Excel)	Yes	No (need Java)	Yes
Widespread use	Low	High	Medium	Low
<i>Staff and Support Needs</i>				
Learning curve	Low	Medium	Medium	Medium
Complexity	Low	Medium	Medium	Medium
Training available	○	●	◐	○
User guide	○	●	●	●
Instructional videos	○	○	●	○
Technical support	○	●	◐	◐
<i>User Experience</i>				
Copy/paste	●	○	◐	◐
Load/save	●	●	●	●
Import/export	○	●	●	○
Auto-fill	○	◐	●	○

Notes: ● = fully supported, ◐ = partially supported, ○ = not supported.

Exhibit 11-4 summarizes more detailed information about each tool that will impact the user, including:

- Available methodologies;
- How inputs are entered, including the ability to easily change the configuration;
- Available outputs, charts, and automated reports; and
- Specialized features that are not required for the HCM methodologies, but useful depending on the application, including maps, adjustment factors, different scenario analyses, and features that make analyses faster, such as built-in error handling and example files.

Exhibit 11-4 Software Tool Methodological Comparison

Features	PPEAG Tool	HCS	FREEVAL	HCM CALC
<i>HCM Methodology</i>				
PPEAG planning methods	●	○	○	○
HCM segment methods	○	●	●	●
HCM facility methods	○	●	●	●
HCM travel time reliability method	○	●	●	●
HCM managed lane analysis	○	●	●	○
<i>Input/Output Features</i>				
Change spatial/temporal configuration	●	●	●	●
Segment-level outputs	●	●	●	●
Facility-level outputs	●	●	●	●
HCM measures	●	●	●	●
Charts/visualizations (core method)	●	●	●	●
Charts/visualizations (reliability)	○	●	●	●
Charts/visualizations (ATDM)	○	○	●	○
Automated report generation	○	●	○	●
Built-in scenario comparison	○	○	●	○
<i>Additional/Specialized Features</i>				
Load/save and shareable files	●	●	●	●
Facility graphic	○	●	●	○
Map interface (real-world coordinates)	○	○	○	○
Calibration (adjustment factors)	●	●	●	●
CAV analysis	○	●	●	●
Built-in weather adjustments	○	●	●	●
Incident scenario analysis	○	●	●	●
Work zone scenario analysis	○	●	●	●
ATDM method	○	○	●	○
Ramp metering	○	○	●	○
Built-in error handling	●	●	●	●
Example files included	○	●	●	○

Notes: ● = fully supported, ● = partially supported, ○ = not supported.

11.2.6 Matching Analysis Methods to Applications

The first consideration when selecting an analysis method is to think about how the results of the analysis will be used. If the analysis' conclusions and recommendations will be used in making final design decisions about roadway geometry or traffic control, or if the analysis will support a decision-making process such as issuing a permit or making a land-use decision, then a detailed analysis should be performed. On the other hand, if the purpose of the analysis is to identify future needs, develop potential alternatives, or prioritize a list of potential projects, then broad brush and/or screening (i.e., planning)

levels of analysis may be more appropriate.

Once the general level(s) of analysis have been determined, the next consideration is the specific analysis tasks required during a project. Here, the goal is to use project resources efficiently by matching the analysis method for a given task to the precision required for the end result. For example, for a corridor study, a first step might involve analyzing many roadway segments along the corridor to identify existing and potential future problem areas. Later steps will confirm the problems and start to identify potential solutions. Therefore, a broad-brush method might be appropriate to start with, to rule out segments from further analysis that clearly have no operational problems.



Because broad-brush methods produce more approximate results than other methods, the operational threshold selected for ruling out a segment should be more conservative than with a screening or detailed method. In other words, when there is doubt about whether a segment would meet an operational standard, the segment should be retained in the analysis.

As the analysis progresses, more detailed methods can be used to confirm the initial results and to begin to identify potential solutions. Although these methods require more input data and more time per segment to perform, there are also fewer segments to analyze at this stage. Continuing with the example of a corridor study, a screening method could be applied to the segments identified as potential problem areas. Some of the “borderline” segments retained from the initial broad-brush method may be dropped after this stage, while the degree of the operational problem will be quantified for the remaining segments and potential solutions can begin to be developed. If the operation of a given segment cannot be analyzed due to a limitation of the analysis method (e.g., over-capacity operation or the inability of the method to analyze a potential solution), then increasingly detailed methods can be applied as required.

A key advantage of using the HCM-based methods described in Section 11.3 is that they produce consistent results across different analysis levels, if consistent inputs are used. Therefore, as a project progresses from identifying problems to identifying potential solutions to preliminary design, forecasted project outcomes should stay reasonably consistent. Of course, specific performance estimates will change somewhat over time as more detailed information is incorporated into the analysis.

11.3 Freeways and Multilane Highways

This section presents analysis methods for freeway and multilane highway segments and facilities. It begins by summarizing the types of data required for each method, along with potential sources of those data. The section then continues by describing segment-based methods; these methods produce v/c ratios and are a necessary first step for performing a facility analysis. The segment methods are followed by descriptions of facility-based methods; these methods can analyze over-capacity conditions over an extended analysis period and can generate additional performance measures, such as

average travel speed.

Each segment or facility type includes methods for both screening and detailed analysis; the facility sections also provide a broad-brush analysis method. Each method is accompanied by an example demonstrating its use.

Exhibit 11-5 summarizes the input data required to calculate a v/c ratio for each of the methods presented in Section 11.3. Following the table, each input is defined, and one or more data sources identified for it.

Exhibit 11-5 Input Data Required to Calculate v/c Ratios for Freeway and Multilane Highway Analysis Methods

Input Data	Generalized Capacity Tables	PPEAG Simplified Segment*	PPEAG Simplified Facility	HCM7 Basic Segment	HCM7 Merge & Diverge	HCM7 Weave	HCM7 Facility
<i>Volume-Related Inputs</i>							
Mainline volume**	●	●	●	●	●	●	●
Peak hour factor		○	○	●	●	●	
Percent heavy vehicles		○	○	●	●	●	●
Terrain class or specific grade	●	●	●	●	●	●	●
Area type (urban/rural)	●						
Section/segment length			●				●
Ramp volume(s)**		●	●		●		●
Acceleration/deceleration lane length					●		●
Distances to adjacent ramps***					●		●
Volumes on adjacent ramps***					●		●
Weaving volumes**		●	●			●	●
<i>Capacity-Related Inputs</i>							
Mainline free-flow speed (FFS)†		●	●	●	●	●	●
Number of mainline lanes	●	●	●	●	●	●	●
Driver population		○	○	○	○	○	○
Ramp metering		○	○		○	○	○
Ramp FFS					●		●
Number of ramp lanes					●		●
Number of weaving lanes						●	●
Weave length		●	●			●	●
Jam density							○
Queue discharge capacity drop							○
<i>CAV-Related Input††</i>							
Percent CAVs	○	○	○	○	○	○	○

Notes: ● = required input, ○ = optional input (can be defaulted), empty = not required.

*Ramp inputs only apply to ramp sections; weave inputs only apply to weave sections.

**HCM facility method requires volumes by 15-minute analysis period.

***Six-lane facilities only (three lanes in each direction).

†If not observed in the field or estimated from the speed limit, will require additional geometric data to estimate, such as lane width, shoulder clearance, and interchange density.

††Freeways only.

Volume-Related Inputs

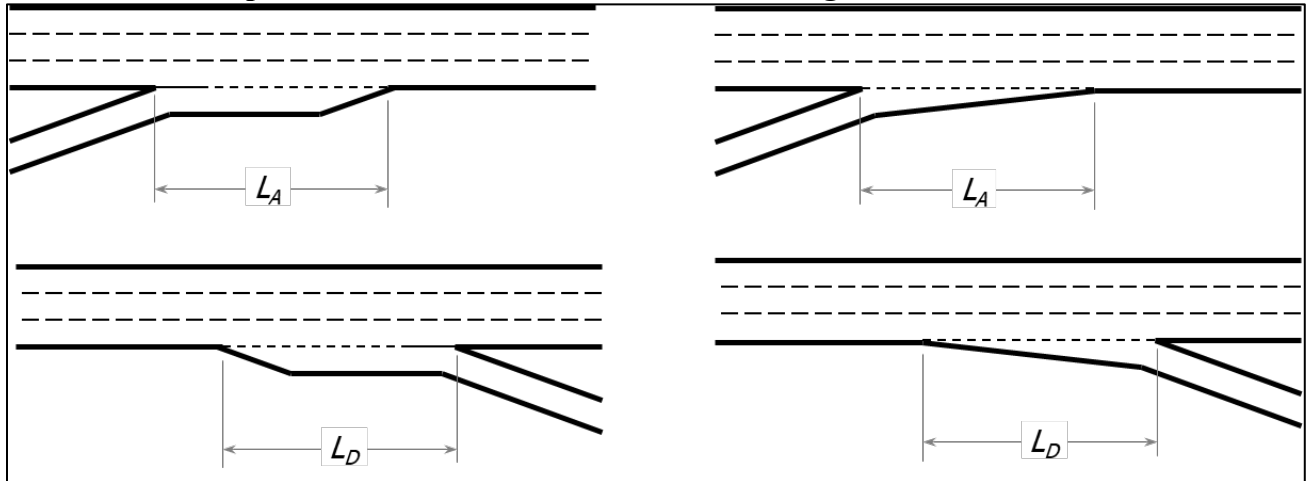
- **Mainline and ramp volumes.** Segment-based analyses require hourly volumes, which can be derived from annual average daily traffic (AADT) volumes, design hour (K) factors, and directional (D) factors for broad-brush and screening

analyses, but preferably from shorter-duration (e.g., 5-or 15-minute) counts for detailed analyses. ODOT's TransGIS system (<http://gis.odot.state.or.us/transgis/>) provides AADTs and *K* and *D* factors for roadway and ramp sections; see Section 3.4 for additional sources of traffic count data. Freeway facility analyses require volumes for each 15-minute period of an analysis hour. The screening method for freeway facilities can apply a default traffic profile to generate 15-minute volumes from a known hourly volume.

- **Peak hour factor (PHF).** The hourly volume divided by four times the peak 15-minute volume. Screening analyses can apply a default value (see Appendix 11C), while detailed analyses will preferably calculate a PHF from a location-specific count.
- **Percent heavy vehicles.** The volume of trucks and buses, divided by the total traffic volume. If a segment contains a steep upgrade (see terrain class discussion below), then the mix of single-unit and tractor-trailer trucks (e.g., 30%/70%) is also required. The percent heavy vehicles can be obtained by adding the truck and bus percentages from TransGIS. The HCM 7th Edition contains lookup tables for grade adjustment factors as a function of percent grade, length of grade, heavy vehicle percentage, and different single-unit and tractor-trailer fleet mixes. Note that the HCM calls out recreational vehicles (RVs) as a separate type of heavy vehicle, but that ODOT uses the FHWA vehicle classification system, in which RVs are classified as either light trucks or single-unit trucks, depending on the number of axles.
- **Terrain class (level, rolling, mountainous) or specific grade.** Level terrain is any terrain (including downgrades) where trucks operate at or near the posted speed. Rolling terrain contains upgrades that cause trucks to slow, but not to their crawl speed. Mountainous terrain is only used in broad-brush and screening applications and contains upgrades where trucks operate at their crawl speed. For detailed analyses, any segment that contains a grade that is either (1) between 2–3% and longer than ½ mile, or (2) steeper than 3% and longer than ¼ mile, should be analyzed as a specific grade (where the slope and grade length are required) rather than as a general terrain class. Grade information can be obtained from ODOT's Vertical Grade Report.
- **Area type (urban/rural).** The area type is determined from the FHWA's functional class for the highway segment. This information is available in TransGIS or in the detailed highway inventory.
- **Section or segment length.** See the individual methods for definitions. Potential sources of length data, depending on the segment type, include recent scaled aerial photos and the detailed highway inventory.
- **Ramp acceleration/deceleration lane length.** The distance from the ramp gore point to the ending/starting point of the taper (see Exhibit 11-6). This information can be determined from recent scaled aerial photos or design drawings.

- **Distances to adjacent ramps.** The distance from the ramp gore point to the gore points of the next upstream and downstream ramp. This information can be obtained from recent scaled aerial photos and the detailed highway inventory.
- **Weaving volumes.** See Section 11.3.3 for guidance.

Exhibit 11-6 Ramp Acceleration and Deceleration Lane Length Definitions



Source: HCM 7th Edition, Exhibit 14-5.

Notes: L_A = acceleration lane length, L_D = deceleration lane length.

Capacity-Related Inputs

- **Mainline free-flow speed (FFS).** See Appendix 11A for guidance.
- **Number of mainline and ramp lanes.** This information can be obtained from aerial photos, TransGIS, and the detailed highway inventory.
- **Driver population.** This is an optional input that describes the mix of drivers in the overall traffic stream, ranging from all familiar drivers to all unfamiliar drivers. See Appendix 11B for guidance.
- **Ramp metering.** This is an optional input that describes whether ramp meters are in operation during the analysis period and, if so, the ramp metering rate.
- **Ramp FFS.** For off-ramps, the posted ramp advisory speed can be used as the ramp FFS. For on-ramps, the smaller of the mainline FFS minus 10 mph or the design speed of the controlling curve on the ramp can be used as the ramp FFS.
- **Number of weaving lanes and weaving length.** See Section 11.3.3 for guidance.
- **Jam density and queue discharge capacity drop.** See Section 11.3.4 for guidance.

CAV-Related Input

- **Percent CAVs.** The percentage of vehicles in the freeway traffic stream with operating cooperative adaptive cruise control systems. See Appendix 6B for guidance.

11.3.1 Basic Freeway and Multilane Highway Segments

Screening Analysis Method

Definition of a Basic Section

Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for estimating the v/c ratio of a basic freeway section. A basic freeway section is defined as the portion of a freeway between an off-ramp and the next downstream ramp.

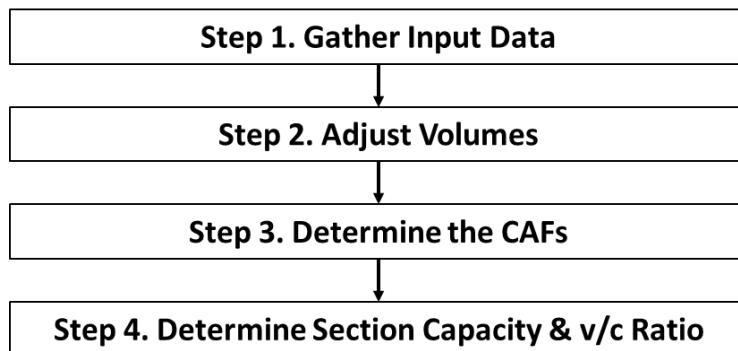
The same basic method, with a slightly different capacity equation, also applies to multilane highway sections. Section boundaries can include signalized intersections, major unsignalized intersections where highway volumes change significantly, lane adds or drops, and changes in terrain type.

Applicability

The method can be applied to basic freeway sections with a FFS ≤ 75 mph and to multilane highways with a FFS ≤ 70 mph. Multilane highways with signal spacing less than 2 miles should be analyzed as urban streets. The capacity adjustment for CAVs can only be applied to freeway sections and not multilane highways.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a basic freeway or multilane highway segment using the screening method:



Step 1: Gather Input Data. The following data are required; inputs in *italics* can be defaulted when not available:

- Hourly demand.
- The *peak hour factor*.
- The *heavy vehicle percentage*.
- The number of lanes.
- The *free-flow speed*.
- The terrain class (level, rolling, mountainous).

See Appendix 11C for Oregon-specific default values. The FFS can be estimated using the “roadway characteristics” method described as part of the detailed analysis method below, or as the speed limit plus 5 mph. See Appendix 11A for adjusting the FFS for differential truck speed limits or for mountainous terrain.

Step 2. Adjust Volumes. The traffic volume is converted to a 15-minute flow rate by dividing the volume by the peak hour factor.

Step 3. Determine the Capacity Adjustment Factors. Optionally, a capacity adjustment factor for driver population CAF_{pop} can be applied, as described in Appendix 11B. Otherwise, a default value of 1.00 is used for this factor. An optional capacity adjustment factor for CAVs CAF_{CAV} can also be applied to freeway sections, as described in Appendix 6B. Otherwise, a default value of 1.00 is used for this factor.

Step 4. Determine the Section Capacity and v/c Ratio. The capacity of a basic freeway section is determined using the following equation (derived from PPEAG Equation 16):

$$c = \frac{(2,200 + 10 \times (\min(70, FFS) - 50))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop} \times CAF_{CAV}$$

where

c = basic freeway section capacity (veh/h);

FFS = section free-flow speed (mph);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV$ = heavy vehicle percentage (e.g., 6% = 6);

N = number of lanes (integer);

CAF_{pop} = driver population capacity adjustment factor; and

CAF_{CAV} = CAV capacity adjustment factor.

The corresponding equation for a basic multilane highway section is:

$$c = \frac{(1,900 + 20 \times (\min(65, FFS) - 45))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop}$$

The v/c ratio is then the flow rate divided by the section capacity.

Example 11-1 Multilane Highway Analysis (Screening Method)

Step 1. Gather Input Data. The multilane highway section being analyzed is located in a rural area with level terrain and has a FFS of 70 mph. There are two lanes in the analysis direction. The AADT is 26,900 with $K=10.0$ and $D=55$, and the volume includes 9.2% heavy vehicles. The driver population is familiar with the facility.

The AADT must be converted into a peak-hour volume by multiplying by the decimal version of the facility's K - and D -factors, resulting in a (rounded) volume of 1,480 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rate is determined by dividing the peak hour volume by the peak hour factor. The PHF is unknown; therefore, the default value of 0.88 for rural multilane highways is used (see Appendix 11C or HCM 7). The resulting demand flow rate is $1,480 / 0.88 = 1,682$ veh/h.

Step 3. Determine the Capacity Adjustment Factor. Because this section has a population of drivers familiar with the facility, $CAF_{pop} = 1.00$.

Step 4. Determine Section Capacity and v/c Ratio. Because the section is located in level terrain, a truck equivalency of 2 is used. The capacity of the basic multilane highway section is then:

$$c = \frac{(1,900 + 20 \times (\min(65, FFS) - 45))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop}$$
$$c = \frac{(1,900 + 20 \times (\min(65, 70) - 45))}{1 + (2 - 1)(9.2/100)} \times 2 \times 1.00 = 4,212 \text{ veh/h}$$

The v/c ratio is then $1,682 / 4,212 = 0.40$.

Example 11-2 Freeway Analysis (Screening Method)

Step 1. Gather Input Data. The freeway segment being analyzed is located in an urban area with mountainous terrain, with a FFS of 55 mph. There are three lanes in each direction. The AADT is 160,000 with $K = 8.2$ and $D = 52$; the volume includes 4.1% heavy vehicles and no CAVs. The driver population is familiar with the facility.

The AADT must be converted into a peak-hour volume by multiplying by the decimal

version of the facility's K - and D -factors, resulting in a (rounded) volume of 6,820 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rate is determined by dividing the peak hour volume by the peak hour factor. The PHF is unknown; therefore, the default value of 0.94 for freeways is used (see Appendix 11C or HCM 7). The resulting demand flow rate is $6,820 / 0.94 = 7,255$ veh/h.

Step 3. Determine the Capacity Adjustment Factors. Because this section has a population of drivers familiar with the facility, $CAF_{pop} = 1.00$. No CAV analysis is being performed; therefore, $CAF_{CAV} = 1.00$.

Step 4. Determine Section Capacity and v/c Ratio. Because the section is located in mountainous terrain, a truck equivalency of 5 is used. The capacity of the basic freeway section is then:

$$c = \frac{(2,200 + 10 \times (\min(70, FFS) - 50))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{pop} \times CAF_{CAV}$$
$$= \frac{(2,200 + 10 \times (\min(70, 55) - 50))}{1 + (5 - 1)(4.1/100)} \times 3 \times 1.00 \times 1.00 = 5,799 \text{ veh/h}$$

The v/c ratio is then $7,255 / 5,799 = 1.25$.

Detailed Analysis Method

Definition of a Basic Segment

Chapter 12 of the HCM provides methods for estimating the v/c ratios of basic freeway and multilane highway segments. Basic segments are any portion of a facility outside the influence area of ramp merges, diverges, and weaving areas, and (for multilane highways) signalized intersections. The influence area of a merge segment extends 1,500 feet downstream from the merge gore point, the influence area of a diverge segment extends 1,500 feet upstream from the diverge gore point, while the influence area of a weaving segment extends 500 feet upstream and downstream from the ramp gore points. The influence area of an isolated signalized intersection on a multilane highway is 1 mile upstream and downstream of the signal, while the influence area of periodic signalized intersections along a multilane highway is 2 miles upstream and downstream. Note that when the signal spacing is less than 2 miles, the roadway is analyzed as an urban street and not as a multilane highway.

Applicability

The method can be applied directly to general-purpose freeway and multilane highway basic segments. Some aspects of the method (e.g., free-flow speed estimation) are also used as part of the analysis of merge, diverge, and weaving segments.

Chapter 12 of the HCM provides an extension to the method for evaluating managed lane basic segments; consult the HCM for details on performing a managed lane analysis.

The method can potentially be applied to extended bridge and tunnel segments by calibrating capacity and speed adjustment factors for those segments (see Appendix 11B).

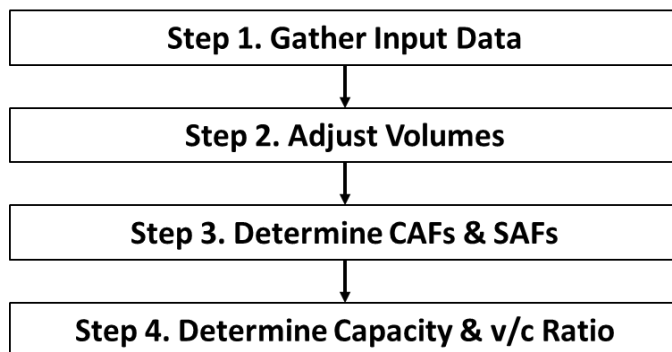
The method is not applicable to facilities with free-flow speeds greater than 75 mph (freeways) or 70 mph (multilane highways), nor is it applicable to freeway segments near toll plazas.



Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a basic freeway or multilane highway segment using the detailed method:



Step 1: Gather Input Data. The following data are required for the segment:

- Hourly demand.
- Peak hour factor.
- Heavy vehicle percentage.
- Free-flow speed (FFS). See Appendix 11A for guidance on determining the FFS.
- Number of lanes.
- Terrain type or specific grade.

Step 2. Adjust Volumes. Traffic volumes are converted from vehicles per hour (veh/h) to passenger cars per hour (pc/h) by applying the following equations, derived from HCM Equation 14-1 and HCM Equation 12-10, respectively:

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

with

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

where

v = demand flow rate under equivalent base conditions (pc/h),

V = hourly volume under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

f_{HV} = heavy vehicle factor (decimal),

P_T = proportion of heavy vehicles in the traffic stream (decimal) (for example: 6% = 0.06), and

E_T = passenger car equivalent (PCE) of one heavy vehicle in the traffic stream.

The PCE of a heavy vehicle can be assumed to be 2 in level terrain and 3 in rolling terrain. However, if an analysis segment contains a grade that is either (1) between 2–3% and longer than ½ mile, or (2) steeper than 3% and longer than ¼ mile, then Exhibits 12-26 through 12-28 in the HCM 7th Edition should be used to determine an appropriate PCE value (see Appendix 11D).

Step 3. Determine Capacity and Speed Adjustment Factors. The HCM provides the ability to adjust capacity and speed to account for non-ideal conditions, through the use of capacity adjustment factors (CAFs) and speed adjustment factors (SAFs). These may be applicable in the following situations:

- To reflect a driver population that includes unfamiliar drivers (see Appendix 11B),
- To calibrate HCM analysis results to match local conditions (see Appendix 11B), or
- To account for the effects of severe weather, incidents, and work zones as part of a travel time reliability analysis (see Section 11.3.7).

Multiple CAFs/SAFs can be included in an analysis (e.g., for driver population and work zones) by multiplying them together to determine an overall CAF/SAF. If no adjustment is to be made, then both CAF and SAF are assigned values of 1.00.

For a capacity analysis of a basic freeway segment, the only CAFs and SAFs that would normally be used initially (i.e., pre-calibration) would be those for driver population.

Step 4. Determine Capacity and v/c Ratio. The per-lane capacity of a basic freeway segment is determined by the following equation, derived from HCM Equations 12-6 and 12-8:

$$c = (2,200 + 10 \times [(FFS \times SAF) - 50]) \times CAF$$

where

c = capacity (pc/h/ln),

FFS = free-flow speed (mph),

SAF = speed adjustment factor (decimal), and

CAF = capacity adjustment factor (decimal).

Similarly, the per-lane capacity of a basic multilane highway segment is determined by the following equation, derived from HCM Equations 12-7 and 12-8:

$$c = (1,900 + 20 \times [FFS - 45]) \times CAF$$



Multilane basic segment capacity is capped at 2,300 pc/h/ln, while freeway basic segment capacity is capped at 2,400 pc/h/ln.

where all variables are as defined previously.

The v/c ratio is then:

$$v/c = \frac{v}{c \times N}$$

where N is the number of directional lanes.

Example 11-3 Multilane Highway Analysis (Detailed Method)

Step 1. Gather Input Data. The multilane highway being analyzed has four lanes (two in each direction), level terrain, 9.2% heavy vehicles, a peak-direction volume of 1,480 veh/h, and a PHF of 0.88. The driver population consists of drivers familiar with the facility. The speed limit for autos is 65 mph, while the speed limit for trucks is 60 mph.

Because the highway has different auto and truck speed limits, a weighted average FFS must be calculated, as described in Appendix 11A. In the absence of detailed roadway characteristics, the auto FFS is estimated as the auto speed limit (65 mph) plus 5 mph, or

70 mph. The truck FFS is estimated as the auto FFS (70 mph) minus the difference in the auto and truck speed limits (5 mph), or 65 mph. The weighted average FFS is then:

$$FFS = (1 - P_T)FFS_{auto} + (P_T)FFS_{truck} = (1 - 0.092)(70) + (0.092)(65) = 69.5 \text{ mph}$$

Step 2. Adjust Volumes. First, calculate the heavy vehicle factor. In level terrain, a heavy vehicle's PCE is 2:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.092(2 - 1)} = \frac{1}{1.092} = 0.916$$

The demand flow rate is then:

$$v = \frac{V}{PHF \times f_{HV}} = \frac{1,480}{0.88 \times 0.916} = 1,836 \text{ pc/h}$$

Step 3. Determine Capacity Adjustment Factors. No capacity adjustment is required, because the driver population is familiar with the facility; therefore CAF is set at 1.00. SAFs do not apply to multilane highways.

Step 4. Determine Capacity and v/c Ratio. The per-lane capacity of the basic multilane highway segment is:

$$c = (1,900 + 20 \times [FFS - 45]) \times CAF$$

$$c = (1,900 + 20 \times [69.5 - 45]) \times 1.00 = 2,390 \rightarrow 2,300 \text{ pc/h/ln}$$

Because the calculated value of 2,390 pc/h/ln exceeds the maximum allowed value of 2,300 pc/h/ln, the per-lane capacity is set at 2,300 pc/h/ln.

The v/c ratio is then:

$$\frac{v}{c} = \frac{v}{c \times N} = \frac{1,836}{2,300 \times 2} = 0.40$$

Example 11-4 Freeway Analysis (Detailed Method)

Step 1. Gather Input Data. The freeway being analyzed climbs a winding 3¼-mile, 6% grade. The design speed of the most severe curve on the grade is 50 mph and there are three lanes in the uphill direction. The uphill volume is 790 veh/h, with 47.7% heavy vehicles (20% single-unit and 80% tractor-trailers) and a PHF of 0.88. The driver population is a balanced mix of familiar and unfamiliar drivers.

Step 2. Adjust Volumes. First, the heavy vehicle factor is calculated. Because this segment contains a specific grade, the PCE is determined from HCM Exhibits 12-26 through 12-28. Exhibit 12-26 is selected because its truck mix (30% single-unit, 70% tractor-trailer) is closest to the observed conditions. For the combination of grade slope, grade length, and heavy vehicle percentage, the PCE is 3.14.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.477(3.14 - 1)} = \frac{1}{2.021} = 0.495$$

The demand flow rate is then:

$$v = \frac{V}{PHF \times f_{HV}} = \frac{790}{0.88 \times 0.495} = 1,814 \text{ pc/h}$$

Step 3. Determine Capacity and Speed Adjustment Factors. Because this segment has a mix of familiar and unfamiliar drivers, Exhibit 26-9 from Volume 4 of HCM 7 is used to obtain the following adjustment factors for driver population: $CAF_{pop} = 0.939$ and $SAF_{pop} = 0.950$. Also refer to Figure 1 of Appendix 11B.

Furthermore, because this segment is considered mountainous terrain, the procedure in Section 3 of Chapter 26 is required to estimate a mixed-flow CAF_{mix} and SAF_{mix} . CAF_{mix} is determined using HCM Equations 26-1 through 26-4. The required inputs are the following:

- The auto-only CAF, CAF_{ao} , which equals CAF_{pop} if no other capacity adjustments (e.g., weather, incidents) are being used in the analysis.
- The heavy-vehicle percentage, expressed as a decimal (e.g., 0.477).
- The grade length (in miles) and the grade percent (as a decimal).

The resulting CAF_{mix} is 0.473.

SAF_{mix} is determined with HCM Equations 26-6 through 26-15, using the following inputs:

- The demand flow rate v and CAF_{mix} , previously calculated.
- The base FFS, in this case, the design speed of the most severe curve, 50 mph.
- The percentage of non-heavy vehicles; the combined percentage of single-unit trucks (SUTs), buses, and RVs; and the percentage of tractor trailers (TTs) (decimal). These values should total 1.000.
- The grade length (in miles) and the grade percent (as a decimal).

The resulting SAF_{mix} is 0.789.

Step 4. Determine Capacity and v/c Ratio. The per-lane capacity of the basic freeway segment is:

$$c = (2,200 + 10 \times [(FFS \times SAF) - 50]) \times CAF \leq 2,400$$
$$c = (2,200 + 10 \times [(50 \times 0.789) - 50]) \times 0.473 = 991 \text{ pc/h/ln}$$

The v/c ratio is then:

$$\frac{v}{c} = \frac{v}{c \times N} = \frac{1,814}{991 \times 3} = 0.61$$

11.3.2 Merge and Diverge Segments

Screening Analysis Method

Definition of a Merge–Diverge Section

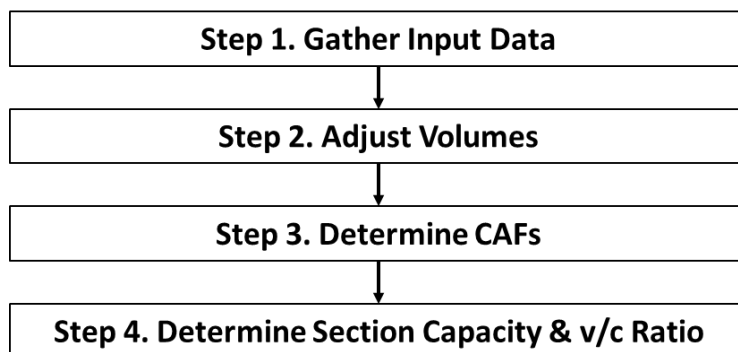
Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for estimating the v/c ratio of a merge–diverge section (called a “ramp section” in the PPEAG). A merge–diverge section is defined as the portion of a freeway between an on-ramp and the next downstream ramp, where the two ramps are not connected by an auxiliary lane.

Applicability

The method can be applied directly to merge–diverge sections on a freeway or multilane highway mainline. Ramp metering can be addressed through a capacity adjustment factor, as described in the detailed method above. The capacity adjustment for CAVs can only be applied to freeway sections and not to multilane highways.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a merge–diverge section using the screening method:



Step 1: Gather Input Data. The following data are required; inputs in *italics* can be defaulted when not available:

- Hourly demands for the freeway entering the merge–diverge section, the on-ramp, and (if present) the off-ramp.
- The *peak hour factor*.
- The *heavy vehicle percentage* within the merge–diverge section.
- The number of lanes on the freeway entering, exiting, and within the merge–diverge section; the number of lanes on the on-ramp; and (if present) the number of lanes on the off-ramp.
- The *free-flow speed* of the freeway mainline.
- The terrain type (level, rolling, mountainous).

See Appendix 11C for Oregon-specific default values.

Step 2. Adjust Volumes. Freeway and ramp volumes are converted to 15-minute flow rates by dividing the volumes by the peak hour factor.

Step 3. Determine Capacity Adjustment Factors. As a default, the capacity of a merge–diverge section is about 95% that of the corresponding basic freeway or multilane highway section; therefore, $CAF_{ramp} = 0.95$. The analyst can substitute a different value if field observations indicate lower capacities. If applicable, capacity adjustment factors for driver population CAF_{pop} (Appendix 11B) and/or ramp metering CAF_{meter} (a value of 1.03) can also be applied. Otherwise, default values of 1.00 are used for these capacity adjustment factors. Finally, an optional capacity adjustment factor for CAVs CAF_{CAV} can be applied to freeway sections, as described in Appendix 6B. Otherwise, a default value of 1.00 is used for this factor.

Step 4. Determine the Merge–Diverge Section Capacity. Capacity is determined using the following equation (derived from PPEAG Equation 16):

$$c = \frac{(2,200 + 10 \times (\min(70, FFS) - 50))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{ramp} \times CAF_{pop} \times CAF_{meter} \times CAF_{CAV}$$

where

c = merge–diverge section capacity (veh/h);

FFS = section free-flow speed (mph);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV$ = heavy vehicle percentage (e.g., 6% = 6);

N = number of lanes in the ramp section (integer);

CAF_{ramp} = ramp section capacity adjustment factor (decimal);

CAF_{pop} = driver population capacity adjustment factor;

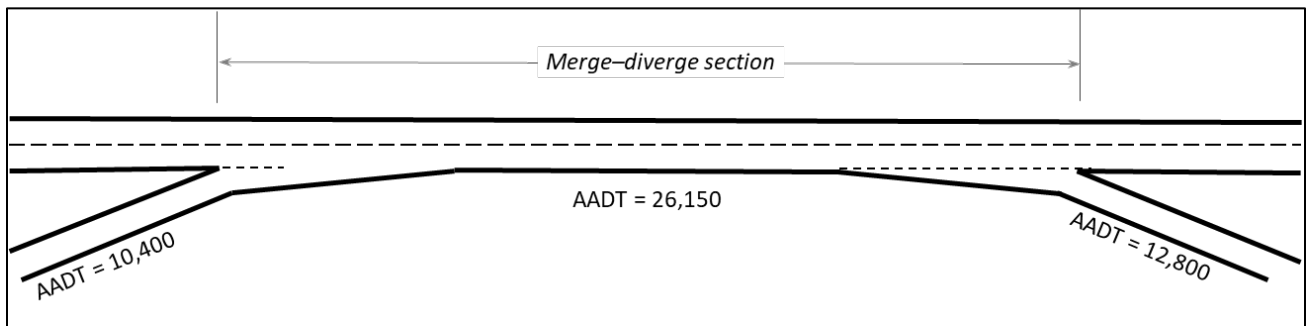
CAF_{meter} = ramp metering capacity adjustment factor; and

CAF_{CAV} = CAV capacity adjustment factor.

The v/c ratio is then the total flow rate in the merge–diverge section divided by the section capacity. Conditions external to the merge–diverge section may also create capacity constraints. The capacity of the freeway mainline can be compared to the entering and exiting demand flows, using the screening procedure in Section 11.3.2. The PPEAG recommends using a value of 2,000 veh/h/ln as a planning-level ramp capacity.

Example 11-5 Merge–Diverge Section Analysis (Screening Method)

Step 1. Gather Input Data. The merge–diverge section has the lane configuration and directional AADTs shown below:



This section of freeway is level and has 16.8% heavy vehicles, a K -factor of 9.3, and a FFS of 60 mph. The ramps have K -factors of 10.0. The on-ramp has 10.2% heavy vehicles, while the off-ramp has 3.5% heavy vehicles. No ramp metering is in use, drivers are familiar with the facility, and there are no CAVs.

The directional AADTs must be converted into peak-hour volumes by multiplying by the decimal version of the facility's K -factor. This results in a (rounded) freeway merge–diverge section volume of 2,430 veh/h, an on-ramp volume of 1,040 veh/h, and an off-ramp volume of 1,280 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rates are determined by dividing the peak hour volumes by the PHF. The PHF is unknown; therefore, the freeway default value of 0.95 is used. For the freeway ramp section, this is $2,430 / 0.95 = 2,558$ veh/h. Similarly, the on-ramp flow rate is 1,095 veh/h and the off-ramp flow rate is 1,347 veh/h.

Step 3. Determine Capacity Adjustment Factors. The merge–diverge section capacity adjustment factor CAF_{ramp} is 0.95. Because the driver population consists of familiar drivers, $CAF_{pop} = 1.00$. There is no ramp metering; therefore, $CAF_{meter} = 1.00$. No CAV analysis is being performed; therefore $CAF_{CAV} = 1.00$.

Step 4. Determine the Section Capacity and v/c Ratio. The section capacity is calculated as:

$$c = \frac{(2,200 + 10 \times (\min(70,60) - 50))}{1 + (2 - 1)(16.8/100)} \times 2 \times 0.95 \times 1.00 \times 1.00 \times 1.00 = 3,741 \text{ veh/h}$$

The section's v/c ratio is then $2,558 / 3,741 = 0.68$.

The on-ramp v/c ratio is $1,095 / 2,000 = 0.55$, while the off-ramp v/c ratio is $1,347 / 2,000 = 0.67$.

Detailed Analysis Method

Definition of Merge and Diverge Segments

Chapter 14 of the HCM provides a method for estimating the v/c ratio of merge and diverge segments—that is, sections of freeway containing either an on-ramp (merge) or an off-ramp (diverge), and where adjacent on- and off-ramps are not connected by an auxiliary lane (those are analyzed as weaving segments). Interchanges where mainline roadways join or split are also treated as merge or diverge segments; these are known as *major merge* and *major diverge segments*.

Merge segments generally extend 1,500 feet downstream from the on-ramp gore point, while diverge segments generally extend 1,500 feet upstream from the off-ramp gore point. If another ramp is located within this influence area, the segment length is reduced and the lower of the two capacity results from the two ramps is applied to the overlap area.

Applicability

The method can be applied directly to:

- Ramps on a freeway mainline;
- Ramps on a multilane highway, if located sufficiently far away from a traffic signal so as not to experience platooning effects; and
- Ramps on collector–distributor roads.

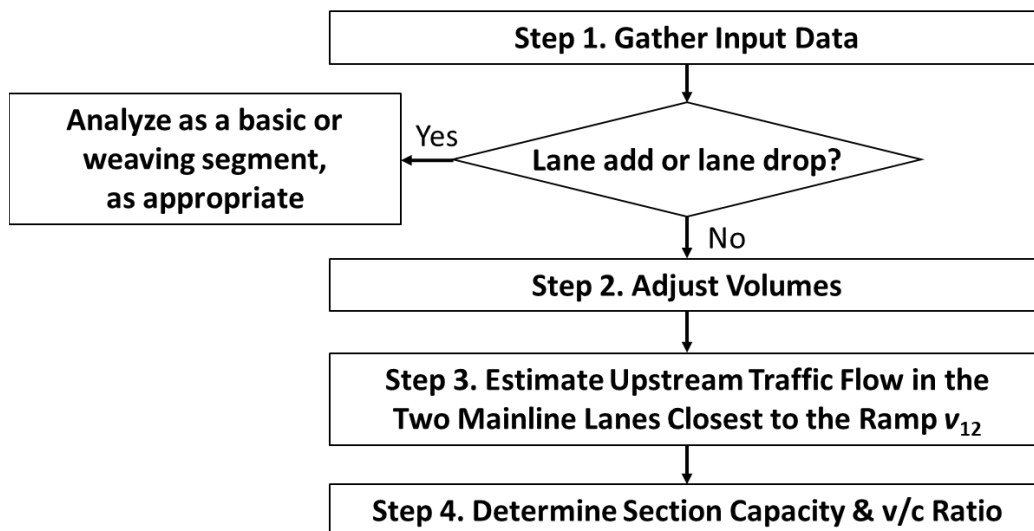
Guidance is provided later in this section on applying this method to on-ramps where ramp metering is in operation and to major merge/diverge areas. Chapter 14 of the HCM also provides an extension to the method for evaluating direct ramps to and from managed lanes (e.g., high-occupancy vehicle or high-occupancy toll lanes). Consult the HCM for details on performing a managed lane merge or diverge analysis.



Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a merge or diverge segment using the detailed method:



Step 1: Gather Input Data. The following data are required for both the freeway (or multilane highway) mainline and the ramp:

- Hourly demands.
- Peak hour factor.
- Heavy vehicle percentage.
- Free-flow speed (FFS). See Appendix 11A for how to determine the mainline FFS and Section 11.3.1 for how to determine the ramp FFS.
- Terrain type (level, rolling) or specific grade.

The following additional data are also required:

- Ramp acceleration/deceleration lane length, as described in Section 11.3.1.
- Number of lanes on the mainline entering the segment, on the ramp, and at the merge/diverge point.
- For six-lane facilities only (three lanes per direction), the distances to the next upstream and downstream ramp, and the volumes on those ramps.

If an on-ramp acceleration lane exceeds 1,500 ft in length, a lane-add situation exists. Similarly, if an off-ramp deceleration lane exceeds 1,500 ft, a lane-drop situation exists. In either situation, the remaining steps in this section are not performed. Instead, the segment is analyzed either as a weaving segment or as a basic segment according to the following rules:

- An added lane that is dropped at the next off-ramp, with no other ramps located in between, is treated as a weaving segment (see Section 11.3.4).
- All other lane-add and lane-drop situations are analyzed as basic segments, with the lane being added or dropped being counted as one of the freeway lanes. Although analyzed as a basic segment, the segment should still be coded in software as a merge or diverge segment, as appropriate, to properly account for the change in volume at the location.
- An added lane that is dropped at a nearby downstream interchange, with other ramps located between the add and drop points, may form a multiple weaving segment. These are discussed in Section 11.3.4. Note that weaving turbulence may exist within a multiple weaving segment that is not fully accounted for by the methodology.

Step 2. Adjust Volumes. Mainline and ramp volumes are converted from vehicles per hour (veh/h) to passenger cars per hour (pc/h) by applying the following equations, derived from HCM Equation 14-1 and HCM Equation 12-10, respectively:

$$v_i = \frac{V_i}{PHF \times f_{HV}}$$

with

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

where

v_i = demand flow rate for movement i under ideal conditions (pc/h),

V_i = hourly volume for movement i under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

f_{HV} = heavy vehicle factor (decimal),

P_T = proportion of heavy vehicles in the traffic stream (decimal) (for example: 6% = 0.06), and

E_T = passenger car equivalent (PCE) of one heavy vehicle in the traffic stream.

The PCE of a heavy vehicle is 2 in level terrain and 3 in rolling terrain. Exhibits 12-26 through 12-28 in the HCM 7th Edition (see also Appendix 11D) can be used to determine an appropriate PCE value when either (1) a grade is 2–3% and longer than ½ mile, or (2) a grade is steeper than 3% and longer than ¼ mile.

Step 3. Estimate Upstream Traffic Flow in the Two Mainline Lanes Closest to the Ramp v_{12} . In most cases, the ramp will be on the right side of the freeway or multilane highway, and the two mainline lanes closest to the ramp will be the two rightmost mainline lanes. For a left-hand ramp, these will be the two leftmost mainline lanes. On a four-lane facility (two lanes in each direction), v_{12} is simply the total demand flow on the mainline just upstream of the ramp influence area. On wider facilities, the process described in the HCM 7th Edition starting on page 14-15 is used to determine v_{12} , with the potential modifications described in the Special Cases section of the chapter (starting on page 14-30) for two-lane ramps, left-side ramps, and 10-lane freeways.

Step 4. Determine Capacity and v/c Ratio. Capacity is determined for both the ramp segment and the ramp roadway. For a ramp junction, the capacity of the ramp influence area (the two mainline lanes closest to the ramp plus the ramp acceleration or deceleration lane) is 4,600 pc/h for a merge segment and 4,400 pc/h for a diverge segment. For ramp roadways, the capacity values given in Exhibit 14-12 in the HCM 7th Edition are used.

Optionally, the capacities of the mainline and the ramp (but not the ramp influence area) can be adjusted using a CAF, as described previously in Section 11.3.1, Step 3. For an initial (i.e., pre-calibration) capacity analysis, the only CAFs that might normally be used are those for driver population CAF_{pop} (see Appendix 11B) and ramp metering CAF_{meter} (described below). Appendix C further provides defaults for merge, diverge, and weaving segment capacities, which are implemented in software in the form of CAFs. For basic freeway segments, the default CAF is 1.0. All applicable CAFs are multiplied together to create an overall CAF, which is applied as follows (HCM Equation 14-21):

$$c_{mda} = c_{md} \times CAF$$

where

c_{mda} = adjusted merge/diverge segment capacity (pc/h),

c_{md} = unadjusted merge/diverge capacity (pc/h), and

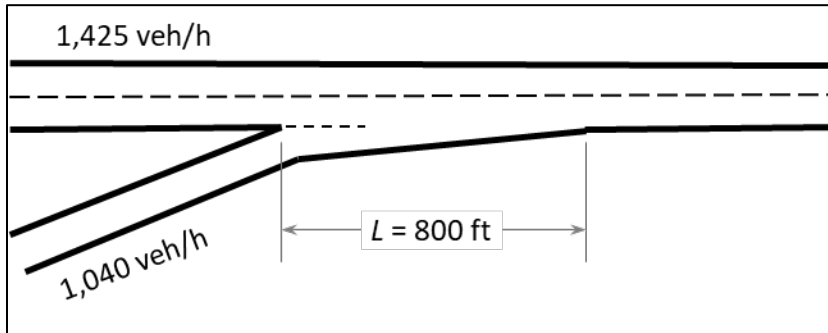
CAF = capacity adjustment factor (decimal), default = 1.00 for basic segments; see Appendix C for other segment types.

The v/c ratio for each location is then determined as follows:

- For a merge ramp junction, the v/c ratio is the sum of v_{12} and the ramp demand flow, divided by 4,600.
- For a diverge ramp junction, the v/c ratio is v_{12} divided by 4,400.
- For the mainline basic segments before or after the merge/diverge segment, the v/c ratio is the mainline demand flow in the upstream or downstream basic segment, divided by the adjusted mainline basic segment capacity.
- For ramp roadways, the v/c ratio is the ramp demand flow divided by the adjusted ramp capacity.

Example 11-6 Merge Segment Analysis, 4-Lane Freeway (Detailed Method)

Step 1. Gather Input Data. The merge segment has the lane configuration and entering volumes shown below:



The freeway mainline is level, with a 70-mph FFS in the merge segment, 16.8% heavy vehicles, a PHF of 0.90, and a driver population familiar with the roadway. The ramp has a free-flow speed of 55 mph, level terrain, 10.2% heavy vehicles, and a PHF of 0.93. There is no ramp meter.

Step 2. Adjust Volumes. First, the heavy vehicle factor is calculated. In level terrain, a heavy vehicle's PCE is 2. Then, for the freeway:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.168(2 - 1)} = \frac{1}{1.168} = 0.856$$

The freeway mainline demand flow rate is then:

$$v_f = \frac{V_f}{PHF \times f_{HV}} = \frac{1,425}{0.90 \times 0.856} = 1,850 \text{ pc/h}$$

Similarly, for the on-ramp, the heavy vehicle factor is 0.907 and the ramp demand flow rate is 1,233 pc/h.

Step 3. Estimate Upstream Traffic Flow in the Two Mainline Lanes Closest to the Ramp v_{12} . This is a four-lane freeway; therefore v_{12} equals the freeway mainline demand flow rate, 1,850 pc/h.

Step 4. Determine Capacity and v/c Ratio. Capacity is determined for the following locations:

- Ramp junction: This is a merge segment, so the ramp junction capacity is 4,600 pc/h.
- Ramp: From HCM Exhibit 14-12, the capacity of a single-lane ramp with a FFS of 55 mph is 2,200 pc/h.

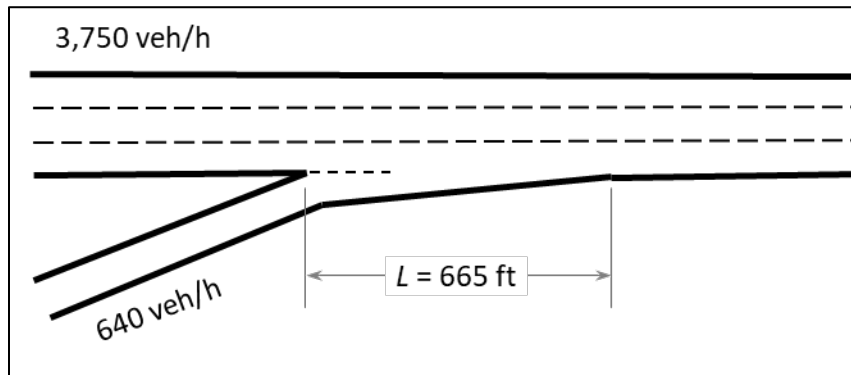
There is no ramp metering and the driver population consists of familiar drivers; therefore, no capacity adjustment is made to either the ramp or the ramp junction.

The v/c ratios are then calculated as follows:

- Ramp junction: The demand flow is the sum of v_{12} and the ramp demand flow. Then, $v/c = (1,850 + 1,233) / 4,600 = 0.67$.
- Ramp: The demand flow is the ramp demand flow. Then, $v/c = 1,233 / 2,200 = 0.56$.

Example 11-7 Merge Segment Analysis, 6-Lane Freeway (Detailed Method)

Step 1. Gather Input Data. The merge segment has the lane configuration and entering volumes shown below:



The freeway mainline is level, with a 70-mph FFS, 17.6% heavy vehicles, a PHF of 0.90, and a driver population familiar with the roadway. The ramp has a FFS of 55 mph, level terrain, 6.5% heavy vehicles, a PHF of 0.88, and no ramp meter. There is an on-ramp 1,800 feet upstream, with a peak-hour volume of 645 veh/h, 6.5% heavy vehicles, level terrain, a PHF of 0.94, and a 45-mph FFS. The next downstream ramp is an off-ramp 9,300 feet away, with a peak-hour volume of 790 veh/h, 14.5% heavy vehicles, level terrain, a PHF of 0.94, and a 45-mph FFS.

Step 2. Adjust Volumes. Volumes are adjusted for heavy vehicles and peak-15-minute conditions in the same manner as in Example 11-6, resulting in following demand flow rates:

- Freeway mainline upstream of the merge: 4,900 pc/h
- On-ramp: 775 pc/h
- Upstream on-ramp: 731 pc/h
- Downstream off-ramp: 962 pc/h

The upstream and downstream ramp flow rates are required because the freeway has a six-lane cross-section and these ramps may influence vehicle positioning in the two right-hand freeway lanes.

Step 3. Estimate Upstream Traffic Flow in the Two Mainline Lanes Closest to the Ramp v_{12} . Because the merge segment has three lanes, the procedures in Chapter 14 of the HCM 7th Edition are applied to determine the traffic flow in the two right-hand freeway lanes. First, the formula used to estimate the proportion of traffic in the two right-hand lanes is determined from HCM Exhibit 14-8. For this situation, where the subject ramp is an on-ramp, the upstream ramp is an on-ramp, and the downstream ramp is an off-ramp, the exhibit says to use either HCM Equation 14-3 or 14-5. The choice of equation is determined by whether the downstream off-ramp is close enough to affect traffic operations. This determination is made by applying HCM Equation 14-7 with the downstream ramp flow rate v_D and the length of the acceleration lane in the merge segment L_A to determine the equilibrium separation distance L_{EQ} of the two ramps:

$$L_{EQ} = \frac{v_D}{0.1096 + 0.000107L_A} = \frac{962}{0.1096 + 0.000107(665)} = 5,322 \text{ ft}$$

The actual distance to the downstream off-ramp, 9,300 ft, is larger than the equilibrium distance; therefore, the HCM instructs the analyst to apply HCM Equation 14-3 to determine the proportion of traffic in the two right-hand lanes P_{FM} :

$$P_{FM} = 0.5775 + 0.000028L_A = 0.5775 + 0.000028(665) = 0.596$$

Finally, HCM Equation 14-2 is used to determine the freeway traffic flow in the two right-most lanes v_{12} :

$$v_{12} = v_F \times P_{FM} = 4,900 \times 0.596 = 2,920 \text{ pc/h}$$

Step 4. Determine Capacity and v/c Ratio. Capacity is determined for the following locations:

- Ramp junction: This is a merge segment, so the ramp junction capacity is 4,600 pc/h.
- Ramp: From HCM Exhibit 14-12, the capacity of a single-lane ramp with a FFS of 55 mph is 2,200 pc/h.

There is no ramp metering and the driver population consists of familiar drivers; therefore, no capacity adjustment is made to either the ramp or the ramp junction.

The v/c ratios are then calculated as follows:

- Ramp junction: The demand flow is the sum of v_{12} and the ramp demand flow. Then, $v/c = (2,920 + 775) / 4,600 = 0.80$.
- Ramp: The demand flow is the ramp demand flow. Then, $v/c = 775 / 2,200 = 0.35$.

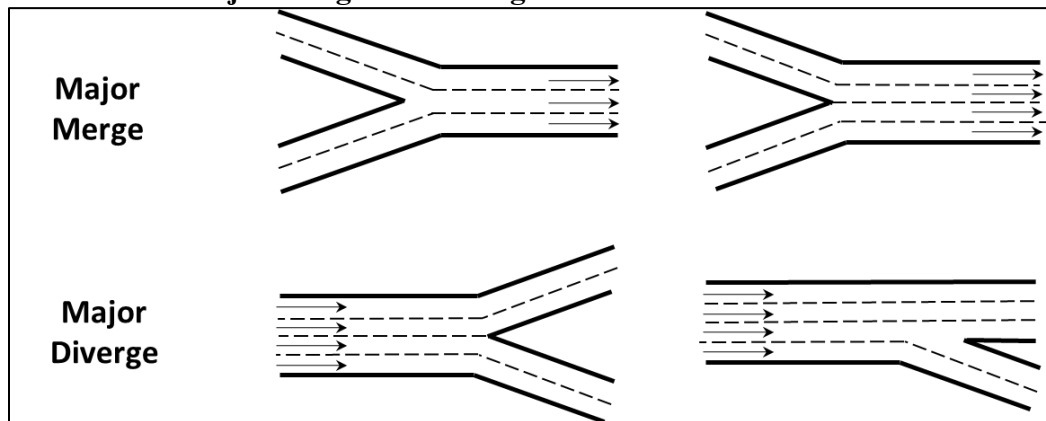
Ramp Metering

The effects of ramp metering on merge and diverge segment capacity can be addressed through a capacity adjustment factor CAF_{meter} . The HCM 7th Edition (Chapter 37, Section 4) recommends a value of 1.03 when ramp metering is in operation. Note that ramp metering will also reduce the per-lane on-ramp capacity to a level equivalent to the metering rate, and therefore the ramp volume entering the freeway may need to be adjusted.

Major Merge and Diverge Areas

The HCM defines a *major merge area* as “one in which two primary roadways, each having multiple lanes, merge to form a single freeway segment,” such as when two freeways join to form a single freeway or a high-speed multilane ramp joins a freeway. The HCM defines a *major diverge area* as “one in which two primary roadways, each having multiple lanes, diverge from a single freeway segment,” such as when a freeway splits into two separate freeways or high-speed multilane ramp diverges from a freeway. Key characteristics of major merge and diverge areas are: (1) all of the roadways involved are at or near freeway design standards, (2) all roadways have two or more lanes, and (3) there is no clear acceleration/deceleration lane. Exhibit 11-7 illustrates potential configurations of major merge and diverge areas.

Exhibit 11-7 Major Merge and Diverge Areas Illustrated



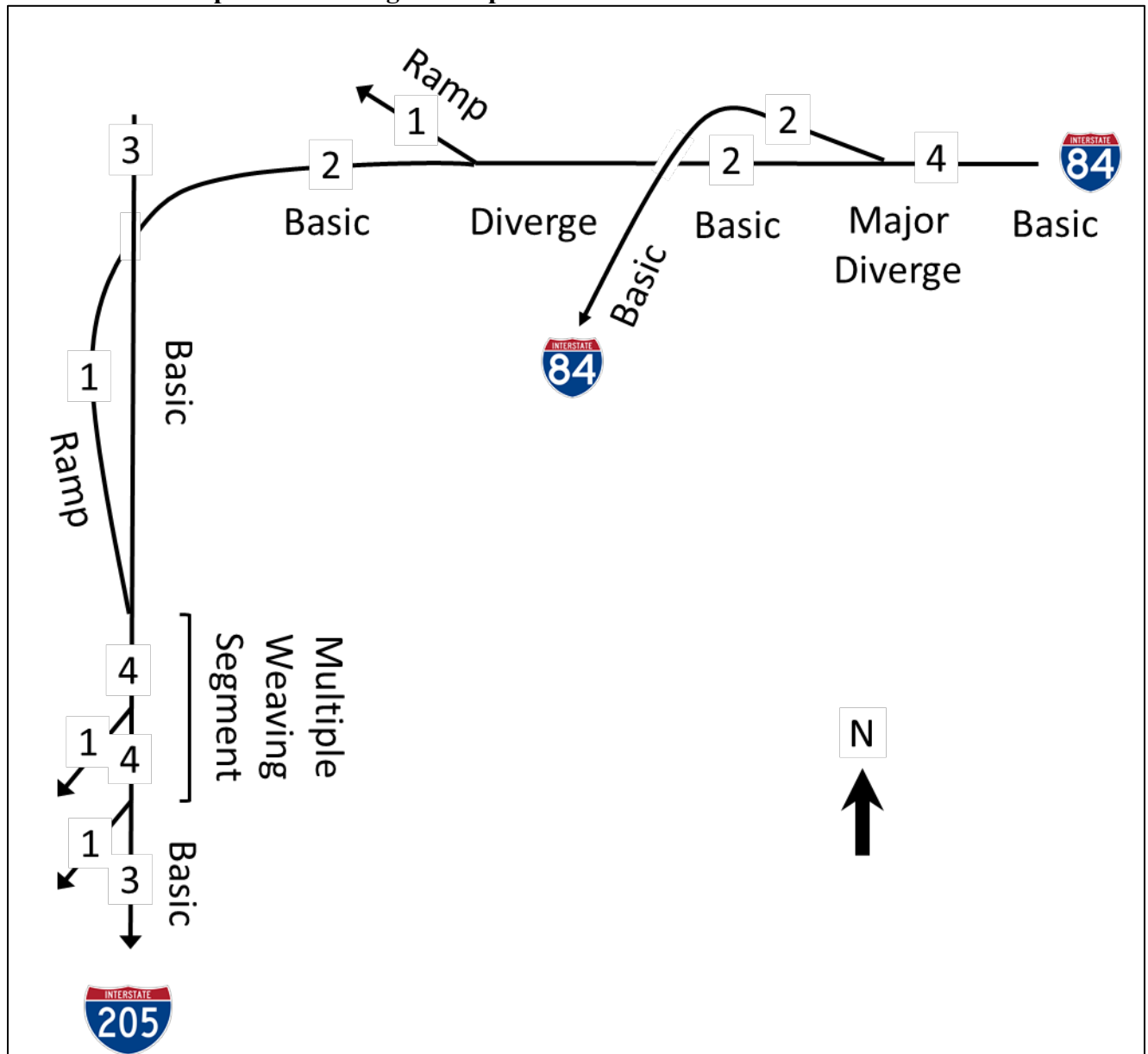
Source: Derived from HCM 7th Edition, Exhibits 14-20 and 14-21.

The HCM recommends evaluating the capacity of major merge and diverge areas by the checking the basic segment capacities of the entry and exit legs. Section 11.3.2 describes how to determine basic freeway segment capacities.

Complex Interchanges

Complex interchanges, with a series of on- and off-ramps, should be broken into smaller basic freeway, (major) merge, (major) diverge, weave, and ramp segments for analysis. Exhibit 11-8 shows an example of I-84 westbound at I-205 and the roadways leading to I-205 southbound.

Exhibit 11-8 Complex Interchange Example: Westbound I-84 to I-205 Southbound



I-84 westbound has four lanes approaching the interchange, which split into two lanes to stay on I-84 and two lanes to connect to both directions of I-205. Because both exit legs of the split have multiple lanes and because both exit legs are designed to freeway standards, the split would be analyzed as a major diverge, involving capacity checks of the entry and exit legs as basic freeway segments.

The exit leg to I-205 has an exit of its own, to I-205 north, with the lanes continuing to I-205 south constructed to freeway standards at the exit. Therefore, the 1,500 feet of roadway preceding the off-ramp would be analyzed as a freeway diverge segment.

Continuing toward I-205 south, one of the lanes is dropped. Following this point, the single-lane roadway would be treated as a ramp roadway, with a lower capacity than a

basic freeway segment. The ramp joins I-205 south as an added lane; therefore, the junction is analyzed as a basic segment rather than as a merge. However, because the added lane is dropped two off-ramps (about 2,800 ft) downstream, a multiple weaving segment exists, which may contain weaving turbulence not accounted for in the methodology. See Exhibit 11-10 in Section 11.3.4 for an example of segmenting a multiple weaving segment into separate merge, diverge, and basic segments.

11.3.3 Weaving Segments

Screening Analysis Method

Definition of a Weaving Section

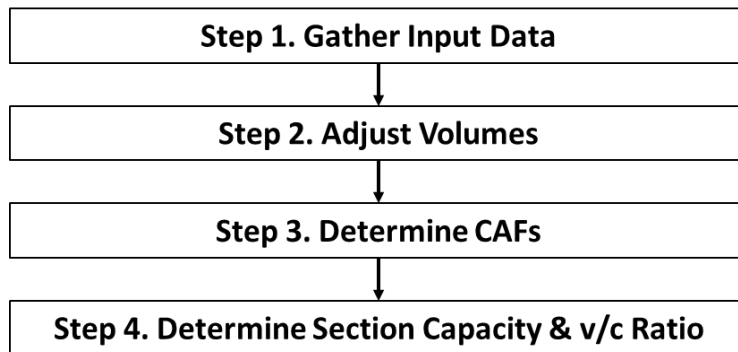
Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for estimating the v/c ratio of a weaving section. A weaving section is defined as the portion of a freeway between an on-ramp and an off-ramp, where the two ramps are connected by an auxiliary lane. Any other combination of ramps should be treated as one or more ramp sections.

Applicability

The method can be applied directly to weaving sections on a freeway or multilane highway mainline. Ramp metering can be addressed through a capacity adjustment factor, as described in the detailed method below. The capacity adjustment for CAVs can only be applied to freeway sections and not to multilane highways.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a weaving section using the screening method:



Step 1: Gather Input Data. The following data are required; inputs in *italics* can be defaulted when not available:

- Hourly demands for the following: freeway entering the weaving section, freeway exiting the weaving section, on-ramp, off-ramp. Weaving volumes should also be used, when available, but for many planning applications they will not be available. Instead, it can be conservatively estimated that there is no ramp-to-

ramp volume and that all of the on- and off-ramp traffic must weave. A regional travel demand model, if available, can be a potential source of future weaving volumes.

- The *peak hour factor*.
- The *heavy vehicle percentage* within the weaving section.
- The weaving section length (i.e., the distance between the ramp gore points).
- The number of lanes on the freeway entering, exiting, and within the weaving section; the number of lanes on the on-ramp; and the number of lanes on the off-ramp.
- The *free-flow speed* of the freeway mainline.
- The terrain type (level, rolling, mountainous).

See Appendix 11C for Oregon-specific default values.

Step 2. Adjust Volumes. The four section entry and exit volumes are converted to 15-minute flow rates by dividing the volumes by the peak hour factor.

Step 3. Determine Capacity Adjustment Factors. The capacity of a weaving section is less than that of a basic freeway section. The following equation (PPEAG Equation 23) is used to estimate the reduction in capacity in a weaving section CAF_{weave} :

$$CAF_{weave} = 0.884 - 0.0752VR + 0.0000243L_{weave}$$

where

CAF_{weave} = weaving section capacity adjustment factor (decimal), always ≤ 1.00 ;

VR = volume ratio (decimal) = weaving flow rate divided by total flow rate in the section; and

L_{weave} = weaving section length (ft).

If applicable, a capacity adjustment factor for driver population CAF_{pop} (Appendix 11B) and/or ramp metering CAF_{meter} (a value of 1.03) can also be applied. Otherwise, a default value of 1.00 is used for these adjustment factors. In addition, an optional capacity adjustment factor for CAVs CAF_{CAV} can be applied to freeway sections, as described in Appendix 6B. Otherwise, a default value of 1.00 is used for this factor.

Step 4. Determine the Weaving Section Capacity. Capacity is determined using the following equation (derived from PPEAG Equation 16):

$$c = \frac{(2,200 + 10 \times (\min(70, FFS) - 50))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{weave} \times CAF_{pop} \times CAF_{meter} \times CAF_{CAV}$$

where

c = weaving section capacity (veh/h);

FFS = section free-flow speed (mph);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV$ = heavy vehicle percentage (e.g., 6% = 6);

N = number of lanes in the weaving section (integer);

CAF_{weave} = weaving section capacity adjustment factor (decimal);

CAF_{pop} = driver population capacity adjustment factor;

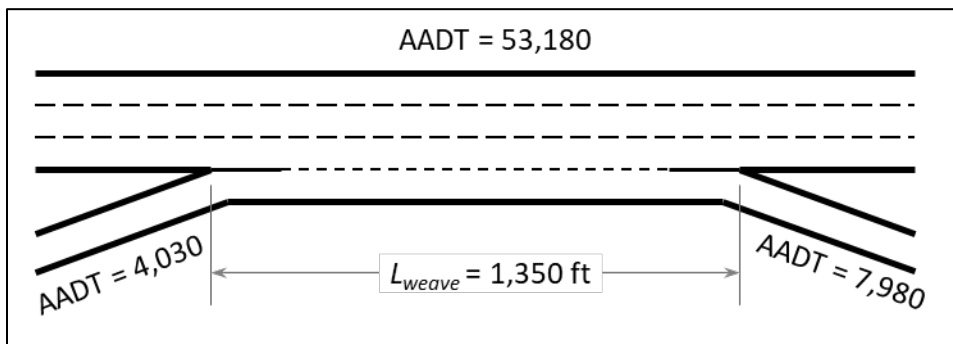
CAF_{meter} = ramp metering capacity adjustment factor; and

CAF_{CAV} = CAV capacity adjustment factor.

The v/c ratio is then the total flow rate in the weaving section divided by the weaving section capacity. Conditions external to the weaving section may also create capacity constraints. The capacity of the freeway mainline can be compared to the entering and exiting demand flows, using the screening procedure in Section 11.3.1, while the capacity of the ramps can be compared to their demand flows using the screening procedure in Section 11.3.2.

Example 11-8 Weaving Section Analysis (Screening Method)

Step 1. Gather Input Data. The weaving section has the lane configuration, length, and directional AADTs shown below:



No weaving volumes are available; therefore, it is assumed conservatively that all ramp volumes must weave. This section of freeway is level and has 6.5% heavy vehicles, no CAVs, a K -factor of 7.6, and a driver population of regular commuters. The freeway has a free-flow speed of 65 mph, the on-ramp has a free-flow speed of 45 mph, and the off-ramp has a free-flow speed of 30 mph. No ramp metering is in use.

The AADTs must be converted into peak-hour volumes by multiplying by the decimal version of the facility's K -factor. This results in a (rounded) on-ramp volume of 305 veh/h, an off-ramp volume of 605 veh/h, and a total weaving section volume of 4,040 veh/h.

Step 2. Adjust Volumes. The peak-15-minute demand flow rates are determined by dividing the peak hour volumes by the PHF. The PHF is unknown; therefore, the freeway default value of 0.95 is used. For the on-ramp, this calculation is $305 / 0.95 = 321$ veh/h. Similarly, the off-ramp flow rate is 637 veh/h and the total section volume is 4,253 veh/h. The volume ratio VR is $(321+637) / 4,253 = 0.223$.

Step 3. Determine Capacity Adjustment Factors. The weaving section capacity adjustment factor is:

$$CAF_{weave} = 0.884 - 0.0752VR + 0.0000243L_{weave}$$

$$CAF_{weave} = 0.884 - 0.0752(0.223) + 0.0000243(1,350)$$

$$CAF_{weave} = 0.900$$

Because the driver population consists of regular commuters, $CAF_{pop} = 1.00$. There is no ramp metering; therefore, $CAF_{meter} = 1.00$. No CAV analysis is being performed; therefore, $CAF_{CAV} = 1.00$.

Step 4. Determine the Weaving Section Capacity and v/c Ratio. The section capacity is calculated as:

$$c = \frac{(2,200 + 10 \times (\min(70,65) - 50))}{1 + (2 - 1)(6.5/100)} \times 4 \times 0.900 \times 1.00 \times 1.00 \times 1.00 = 7,944 \text{ veh/h}$$

The section's v/c ratio is then $4,253 / 7,944 = 0.54$.

The demand entering the weaving section is the section demand minus the on-ramp demand, or 3,932 veh/h, while the capacity of the upstream basic segment is:

$$c = \frac{(2,200 + 10 \times (\min(70,65) - 50))}{1 + (2 - 1)(6.4/100)} \times 3 \times 1.000 \times 1.00 \times 1.00 = 6,626 \text{ veh/h}$$

Therefore, there is no capacity constraint on the upstream segment. Similarly, the demand on the downstream basic segment is 3,616 veh/h, compared to a capacity of 6,626 veh/h, and therefore no capacity constraint exists on the downstream segment. Ramp v/c ratios

can be checked using the same process as illustrated in Example 11-9 for the detailed method.

Detailed Analysis Method

Definition of a Weaving Segment

Chapter 13 of the HCM provides a method for estimating the v/c ratio of a weaving segment. The most common form of weaving segment is a *one-sided weave*, where an on-ramp is closely followed by an off-ramp on the same side of the freeway and:

- The two ramps are connected by an auxiliary lane that drops at the off-ramp; and
- No weaving maneuver requires more than two lane changes.

An uncommon form of weaving segment also addressed by the method is a *two-sided weave*, where either (1) an on-ramp on one side of the freeway is closely followed by an off-ramp on the opposite side of the freeway, or (2) at least one weaving maneuver requires three or more lane changes.

A weaving segment extends 500 feet upstream from the on-ramp gore point and 500 feet downstream from the off-ramp gore point.

Applicability

The method can be applied directly to:

- Weaving segments on a freeway mainline;
- Weaving segments on a multilane highway, if located sufficiently far away from a traffic signal so as not to experience platooning effects; and
- Weaving areas on collector–distributor roads.

The method is potentially applicable, with analyst modifications, to:

- Weaving segments where ramp metering is in operation on the on-ramp, and
- Multiple weaving segments (e.g., an on-ramp connected by an auxiliary lane to two closely spaced downstream off-ramps).

Guidance on these potentially applicable situations is provided below, and simulation is a potential supplemental or alternative tool that can be applied in these cases.

The method cannot be used to evaluate weaving on urban streets, including freeway

frontage roads.

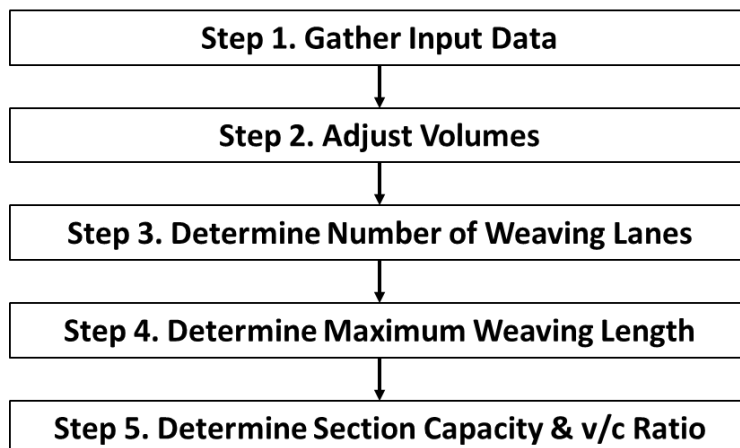
Chapter 13 of the HCM also provides an extension to the method for evaluating weaving associated with managed lanes (e.g., high-occupancy vehicle or high-occupancy toll lanes). These situations include weaving between the managed and general-purpose lanes at managed lane access points, weaving across the general-purpose lanes between general-purpose ramps and the managed lanes, and weaving associated with direct ramps into and out of the managed lanes. Consult the HCM for details on performing a managed lane weaving analysis.



Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.

Method for Calculating the v/c Ratio

The following steps are used to calculate the v/c ratio for a weaving segment using the detailed method:



Step 1: Gather Input Data. The following data are required:

- Hourly demands for the following movements: freeway-to-freeway, freeway-to-ramp, ramp-to-freeway, and ramp-to-ramp. Most often, only volumes entering and exiting the weaving segment will be available. However, if just one of the four weaving movements is counted or estimated from a model (ramp-to-ramp volumes are generally lowest and easiest to count by various methods), the other three movements can be determined from the entry and exit volumes (see Example 11-9 for an illustration of the process).
- The peak hour factor.
- The heavy vehicle percentage within the weaving segment.

- The “short length” of the weaving area, defined as the distance where broken pavement striping permits lane-changing movements.
- The number of lanes on the freeway entering, exiting, and within the weaving segment; the number of lanes on the on-ramp; and the number of lanes on the off-ramp.
- The free-flow speeds of the freeway mainline (see Appendix 11A) and the ramps (see Section 11.3.1).
- The terrain type (level, rolling) or the specific grade.

Step 2. Adjust Volumes. The four movement volumes are converted from vehicles per hour (veh/h) to passenger cars per hour (pc/h) by applying the following equations, derived from HCM Equation 13-1 and HCM Equation 12-10, respectively:

$$v_i = \frac{V_i}{PHF \times f_{HV}}$$

with

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

where

v_i = demand flow rate for movement i under ideal conditions (pc/h),

V_i = hourly volume for movement i under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

f_{HV} = heavy vehicle factor (decimal),

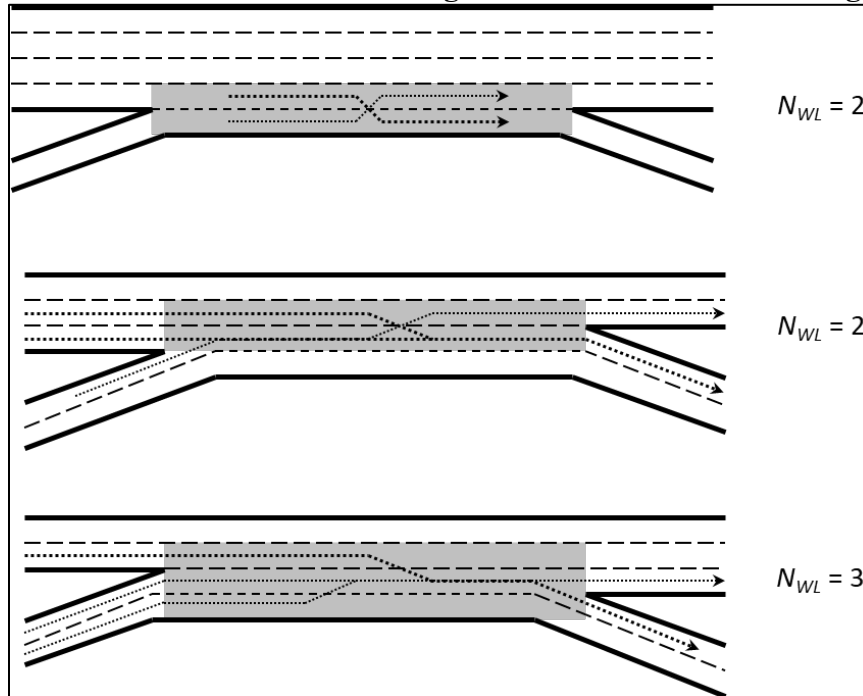
P_T = proportion of heavy vehicles in the traffic stream (decimal) (for example: 6% = 0.06), and

E_T = passenger car equivalent (PCE) of one heavy vehicle in the traffic stream.

The PCE of a heavy vehicle is 2 in level terrain and 3 in rolling terrain. Exhibits 12-26 through 12-28 in the HCM 7th Edition (see also Appendix 11D) can be used to determine an appropriate PCE value when either (1) a grade is 2–3% and longer than ½ mile, or (2) a grade is steeper than 3% and longer than ¼ mile.

Step 3. Determine the Number of Weaving Lanes. The number of weaving lanes N_{WL} in a two-sided weaving segment is 0 by definition. In one-sided weaving segments, N_{WL} is the total number of lanes from which a weaving maneuver can be made with zero or one lane changes. Exhibit 11-9 shows values of N_{WL} for common weaving configurations.

Exhibit 11-9 Number of Weaving Lanes for Common Weaving Configurations



Source: Derived from HCM 7th Edition, Exhibit 13-5.

Step 4. Determine the Maximum Weaving Length. The maximum length of a weaving segment L_{MAX} is determined using the following equation (HCM Equation 13-4):

$$L_{MAX} = [5,728(1 + VR)^{1.6}] - (1,566N_{WL})$$

where

L_{MAX} = maximum weaving segment length (ft),

VR = volume ratio (decimal) = weaving demand flow rate divided by the total demand flow rate in the weaving segment, and

N_{WL} = number of weaving lanes.

If $L_S > L_{MAX}$, segment should be analyzed as separate merge and diverge segments, possible with a basic segment between them depending on distances, follow procedure in Section 11.3.2; otherwise, continue with weaving analysis.

Step 5. Determine the Weaving Segment Capacity and v/c Ratio. The weaving segment reaches capacity when either its density exceeds 43 pc/mi/ln or when the weaving demand flow exceeds a specified value. The capacity of a weaving segment based on density is determined from the following equation (HCM Equation 13-5):

$$c_{IWL} = c_{IFL} - [438.2(1 + VR)^{1.6}] + (0.0765L_S) + (119.8N_{WL})$$

where

c_{IWL} = per-lane weaving segment capacity under equivalent ideal conditions

(pc/h/ln),

c_{IFL} = per-lane capacity of a basic freeway (or multilane highway, as appropriate) segment with the same free-flow speed as the weaving segment, under equivalent ideal conditions (pc/h/ln) (see Section 11.3.2),

VR = volume ratio,

L_S = short length of the weaving segment (ft), and

N_{WL} = number of weaving lanes.

The capacity of a weaving segment based on weaving flow c_{IW} (pc/h) is either $(2,400 / VR)$ when $N_{WL} = 2$, or $(3,500 / VR)$ when $N_{WL} = 3$. No capacity value for weaving flow is defined for two-sided weaving segments; the capacity based on density is used instead.

These two capacity values (based on ideal conditions) are then converted into capacities under prevailing conditions as follows (HCM Equations 13-6 and 13-8):

$$c_W = c_{IWL} \times N \times f_{HV}$$
$$c_W = c_{IW} \times f_{HV}$$

where c_W is the weaving segment capacity under prevailing conditions (veh/h), N is the number of lanes within the weaving area, and all other variables are as defined previously. The lower of the two values of c_W is taken as the capacity of the weaving segment under prevailing conditions.

As described previously in Section 11.3.2, Step 3, capacity can be optionally adjusted using a CAF. For an initial (i.e., pre-calibration) capacity analysis, the only CAFs that might normally be used are those for driver population CAF_{pop} (see Appendix 11B) and ramp metering CAF_{meter} (described below). All applicable CAFs are multiplied together to create an overall CAF, which is applied as follows (HCM Equation 13-9):

$$c_{wa} = c_w \times CAF$$

where

c_{wa} = adjusted weaving segment capacity (veh/h),

c_w = unadjusted weaving segment capacity (veh/h), and

CAF = capacity adjustment factor (decimal), default = 1.00.

Finally, the volume-to-capacity ratio v/c is determined as follows (HCM Equation 13-10):

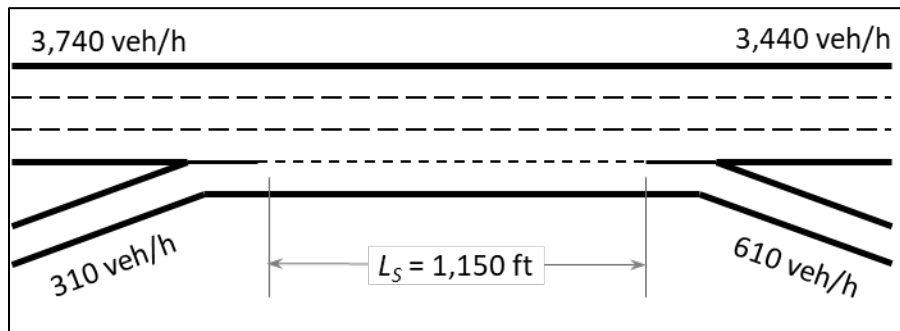
$$v/c = \frac{v \times f_{HV}}{c_{wa}}$$

where all variables have been defined previously. The heavy vehicle adjustment factors are used multiple times because v reflects equivalent ideal conditions, while c_{wa} and any capacity adjustment factors reflect prevailing conditions.

Conditions external to the weaving segment may also create capacity constraints. Therefore, the capacity of the freeway mainline should be compared to the entering and exiting demand flows (see Section 11.3.2) and the capacity of the ramps should be compared to their demand flows (see Section 11.3.3).

Example 11-9 Weaving Segment Analysis (Detailed Method)

Step 1. Gather Input Data. The weaving segment has the lane configuration and entering and exiting volumes shown below:



In addition, 50 veh/h are counted making ramp-to-ramp movements. The remaining weaving movements are determined as follows:

- Ramp-to-freeway = $310 - 50 = 260$ veh/h
- Freeway-to-ramp = $610 - 50 = 560$ veh/h
- Freeway-to-freeway = $3,740 - 560 = 3,180$ veh/h

This section of freeway is a downgrade and has 6.3% heavy vehicles, a driver population of regular commuters, and a PHF of 0.94. The freeway has a free-flow speed of 65 mph, the on-ramp has a free-flow speed of 45 mph, and the off-ramp has a free-flow speed of 30 mph. The on-ramp has 2.7% heavy vehicles, while the off-ramp has 9.1% heavy vehicles.

Step 2. Adjust Volumes. First, the heavy vehicle factor is calculated. In level terrain (which includes most downgrades), a heavy vehicle's PCE is 2. Then:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.063(2 - 1)} = \frac{1}{1.063} = 0.941$$

The freeway-to-freeway demand flow rate is then:

$$v_{FF} = \frac{V_{FF}}{PHF \times f_{HV}} = \frac{3,180}{0.94 \times 0.941} = 3,595 \text{ pc/h}$$

Similarly, the other flow rates are:

- $v_{RR} = 55$ pc/h

- $v_{RF} = 284$ pc/h
- $v_{FR} = 650$ pc/h
- on-ramp = 339 pc/h
- freeway mainline entering = 4,229 pc/h
- freeway mainline exiting = 3,890 pc/h
- off-ramp = 708 pc/h

The total demand flow rate in the weaving segment is $3,595 + 55 + 284 + 650 = 4,584$ pc/h. The volume ratio is the weaving flow divided by the total flow: $(284 + 650) / 4,584 = 0.204$.

Step 3. Determine the Number of Weaving Lanes. By comparison with the uppermost weaving area depicted in Exhibit 11-9, N_{WL} is 2.

Step 4. Determine the Maximum Weaving Length. The maximum length of a weaving segment with this configuration and volume ratio is:

$$L_{MAX} = [5,728(1 + VR)^{1.6}] - (1,566N_{WL}) = [5,728(1 + 0.204)^{1.6}] - (1,566 \times 2) = 4,577 \text{ ft}$$

The short length of this weaving area, 1,150 ft, is less than the maximum length.

Therefore, it is correct to treat the weaving area as a weaving segment and the process continues to Step 5.

Step 5. Determine the Weaving Segment Capacity and v/c Ratio. First, the ideal capacity of an equivalent basic segment with a 65-mph free-flow speed must be determined. From Exhibit 12-7 in the HCM 7th Edition, this value is 2,350 pc/h/ln. The weaving segment's ideal capacity based on density is then:

$$c_{IWL} = c_{IFL} - [438.2(1 + VR)^{1.6}] + (0.0765L_S) + (119.8N_{WL})$$

$$c_{IWL} = 2,350 - [438.2(1 + 0.204)^{1.6}] + (0.0765 \times 1,150) + (119.8 \times 2) = 2,088 \text{ pc/h/ln}$$

The weaving segment's ideal capacity based on weaving flow is $2,400 / 0.204 = 11,765$ pc/h.

These two capacities are then converted into capacities under prevailing conditions as follows:

$$c_W = c_{IWL} \times N \times f_{HV} = 2,088 \times 4 \times 0.941 = 7,859 \text{ pc/h}$$

$$c_W = c_{IW} \times f_{HV} = 11,765 \times 0.941 = 11,071 \text{ pc/h}$$

The lower of these two values, 7,859 pc/h, is taken as capacity. No adjustment to capacity is needed for driver population, therefore $c_{WA} = c_W$. Finally, the v/c ratio is determined to be:

$$v/c = \frac{v \times f_{HV}}{c_{wa}} = \frac{4,584 \times 0.941}{7,859} = 0.55$$

The capacity of the freeway mainline entering and exiting the weaving segment is $(3 \times 2,350) = 7,050$ pc/h, which is greater than the entering and exiting demands. From Exhibit 14-12 in the HCM 7th Edition, the capacity of a single-lane ramp with a 45-mph free-flow speed is 2,100 pc/h, which is greater than the on-ramp demand. Similarly, the capacity of a single-lane ramp with a 30-mph free-flow speed is 1,900 pc/h, which is greater than the off-ramp demand. Therefore, there are no external capacity constraints to address.

Ramp Metering

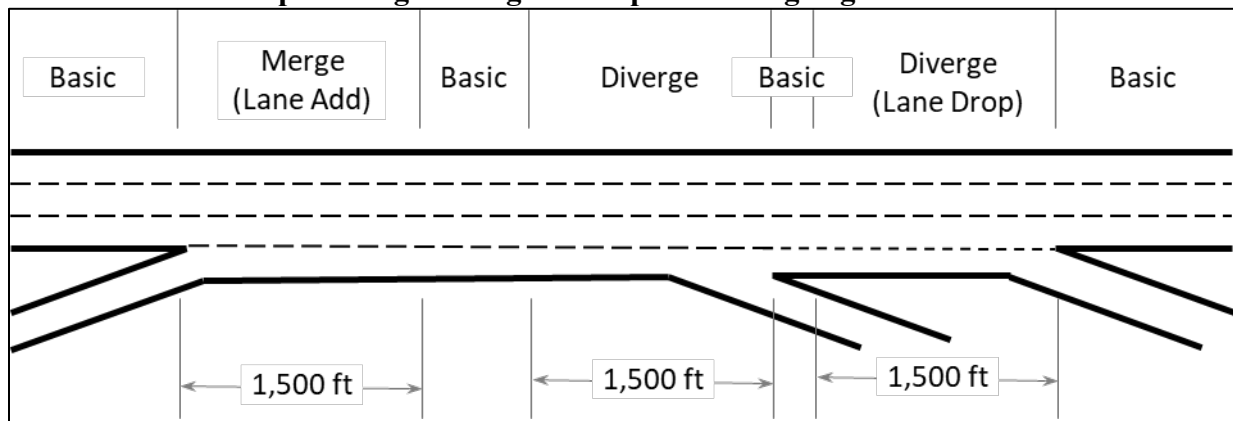
The effects of ramp metering on weaving area capacity can be addressed through a capacity adjustment factor. No specific guidance exists in the HCM for the capacity effects of ramp metering in a weaving segment. In the absence of local data, the HCM capacity adjustment factor CAF_{meter} for merge segments can be used to approximate the effect on weaving segments. The HCM 7th Edition (Chapter 37, Section 4) recommends a value of 1.03 when ramp metering is in operation. Ramp metering will reduce the on-ramp capacity to a level equivalent to the metering rate.

Multiple Weaving Segments

Multiple weaving segments exist when the combination of on- and off-ramps creates weaving movements between multiple sets of origins and destinations. The HCM 7th Edition recommends analyzing these as a series of merge, diverge, basic, and simple weaving segments.

Exhibit 11-10 shows an example of a multiple weaving segment with one on-ramp followed by two off-ramps, all connected by an auxiliary lane. The 1,500 feet downstream of the on-ramp is a merge segment with a lane add. Because of the added lane, the segment is analyzed the same as a basic segment; however, it is coded in software as a merge segment because of the need to account for the volume entering the freeway at this point. The portion of the freeway 1,500 feet upstream of the first off-ramp is analyzed as a diverge segment, while the portion of the freeway between the merge and diverge segments is analyzed as a basic segment. The 1,500 feet of the freeway upstream of the second off-ramp is a diverge segment with a lane drop; it is analyzed as a basic segment but is coded in software as a diverge segment to account for the volume leaving the freeway. Finally, the portion of the freeway between the two diverge segments is analyzed as a basic segment.

Exhibit 11-10 Example of Segmenting a Multiple Weaving Segment



11.3.4 Freeway Facilities

Freeway facility methods evaluate the operations of an extended stretch of freeway. The detailed and screening analysis methods aggregate the results of individual segment or section analyses to generate a range of additional useful performance measures, including average travel speed and vehicle-hours of delay, as well as measures of travel time reliability (discussed in Section 11.3.7). These two methods can be applied to study periods more than one hour long and to freeways operating over capacity. This section also provides a broad-brush facility analysis method, consisting of a table of generalized capacity values that can be adjusted to local conditions, to the extent data (e.g., heavy-vehicle percentage, peak hour factor) are available.

Broad-Brush Analysis Method

Exhibit 11-11 contains a table providing estimates of design-hour, peak-direction freeway capacities. If only AADTs are available, they can be converted to a directional design-hour volume V by applying K - and D -factors as follows:

$$V = AADT \times \frac{K}{100} \times \frac{D}{100}$$

Exhibit 11-11 Generalized Freeway Capacities (veh/h)

Area Type	Terrain	Posted Automobile Speed Limit (mph)				
		50	55	60	65	70
Urban	Level	3,825	3,910	3,995	4,080	4,165
	Rolling	3,655	3,735	3,815	3,895	3,980
	Mountainous	3,350	3,425	3,500	3,570	3,645
Rural	Level	3,215	3,285	3,360	3,430	3,500
	Rolling	2,680	2,740	2,800	2,860	2,915
	Mountainous	2,010	2,055	2,100	2,145	2,190

Note: Assumptions used in this table are: HV% = 5 (urban), 25 (rural); PHF = 0.94; 2 lanes per direction; free-flow speed = posted speed + 5 mph; driver population familiar with the facility; CAF_{ramp} = 0.95.

When specific heavy vehicle percentages and/or peak-hour factors are known, the number of lanes is greater than two, or driver population or CAV effects are desired to be included, the values in Exhibit 11-11 can be adjusted as follows to provide a better estimate of capacity reflecting the local conditions:

$$c_{adj} = c_{table} \times \frac{PHF_{local}}{PHF_{table}} \times \frac{1 + (E_T - 1)(\%HV_{table}/100)}{1 + (E_T - 1)(\%HV_{local}/100)} \times \frac{N_{local}}{2} \times CAF_{pop} \times CAF_{CAV}$$

where

c_{adj} = adjusted facility capacity (veh/h);

c_{table} = capacity value from Exhibit 11-14 (veh/h);

PHF_{local} = local peak hour factor (decimal);

PHF_{table} = peak hour factor assumed in Exhibit 11-14 (decimal);

E_T = truck equivalency = 2 (level terrain), 3 (rolling terrain), or 5 (mountainous terrain);

$\%HV_{table}$ = heavy vehicle percentage assumed in Exhibit 11-14 (e.g., 25% = 25);

$\%HV_{local}$ = local heavy vehicle percentage (e.g., 25% = 25);

N_{local} = number of directional travel lanes;

CAF_{pop} = local capacity adjustment factor for driver population; and

CAF_{CAV} = optional capacity adjustment factor for CAVs from Appendix 6B (default = 1.00).

Example 11-10 Freeway Capacity Analysis (Broad-Brush Method)

A six-lane urban freeway (three lanes in each direction) is located in rolling terrain and has a 50-mph speed limit. The AADT is 121,400, the K -factor is 7.7, the D -factor is 54, the PHF is 0.92, the heavy-vehicle percentage is 9.1, and there are no CAVs.

The design-hour volume V is:

$$V = AADT \times \frac{K}{100} \times \frac{D}{100} = 121,400 \times \frac{7.7}{100} \times \frac{54}{100} = 5,050 \text{ veh/h}$$

The capacity obtained from Exhibit 11-14, which assumes 5% heavy vehicles, a PHF of 0.94, and two travel lanes, is 3,655 veh/h. An adjusted local capacity can be determined as follows by substituting the local heavy-vehicle percentage, PHF, and number of lanes, while keeping the table values for all other inputs that are unknown or unchanged:

$$c_{adj} = c_{table} \times \frac{PHF_{local}}{PHF_{table}} \times \frac{1 + (E_T - 1)(\%HV_{table}/100)}{1 + (E_T - 1)(\%HV_{local}/100)} \times \frac{N_{local}}{2} \times CAF_{pop} \times CAF_{CAV}$$
$$c_{adj} = 3,655 \times \frac{0.92}{0.94} \times \frac{1 + (3 - 1)(5/100)}{1 + (3 - 1)(9.1/100)} \times \frac{3}{2} \times 1.00 \times 1.00 = 4,994 \text{ veh/h}$$

The v/c ratio is then $(5,050 / 4,994) = 1.01$.

Screening Analysis Method

Definition of a Freeway Facility

Section H6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a simplified method for evaluating the performance of a freeway facility or “supersection.” A supersection consists of multiple contiguous freeway sections, extending up to the distance that an automobile can drive at the posted speed in 15 minutes—typically 9 to 12 miles in urban areas and up to 15 miles in rural areas. In addition to the maximum length criterion, other criteria to be considered when defining the endpoints of a supersection include: freeway-to-freeway interchanges, urbanized area boundaries, major intersecting routes, major trip generators (e.g., central city downtowns, major airports), and the state border. Unlike the detailed method, it is not necessary that the first and last segments of the supersection operate below capacity throughout the study period, although the mainline demand entering the first section should be below the section’s capacity throughout the study period. One of the method’s simplifications is that all unserved demand to a section is stored as a vertical queue at the section entrance, rather than being propagated into the upstream section.

Study Period Length

The method was designed for a 1-hour study period, although longer study periods can be

studied if the analyst has (or can generate) 15-minute traffic demands for the analysis periods beyond the peak hour. To capture the full effects of any congestion that occurs, the entire facility should operate below capacity during the first and last 15-minute analysis periods.

Applicability

The method is applicable to general-purpose freeway lanes. All of the screening method limitations described in Sections 11.3.2 through 11.3.4 also apply to freeway facilities. In particular, if the capacity of an off-ramp roadway or terminal is exceeded, causing queues to spill back onto the mainline, the facility method cannot be used and an alternative analysis tool will be required.

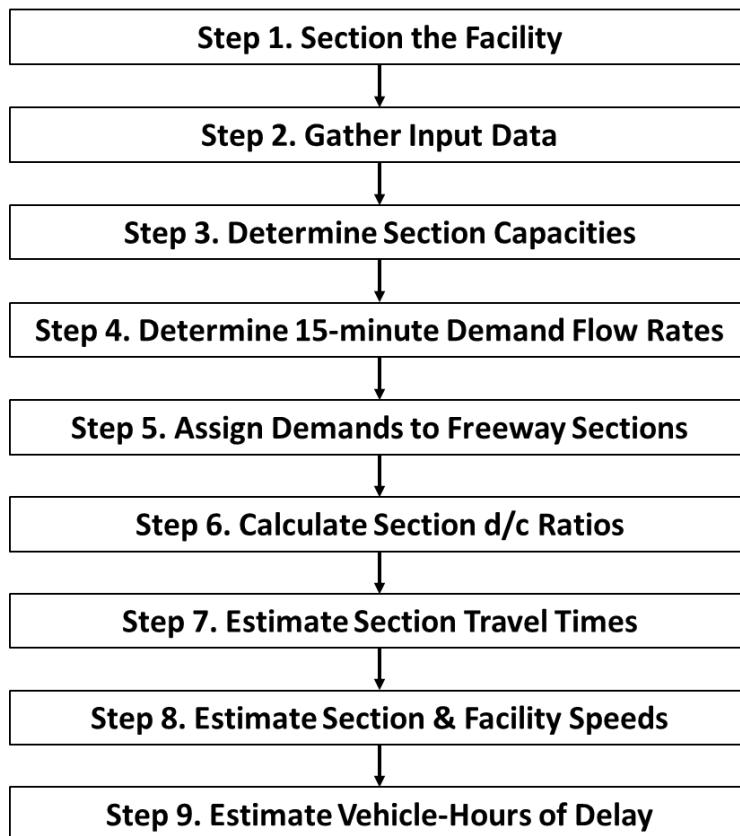
Calculation Process

The method estimates the average travel time required to traverse each freeway section during each 15-minute analysis period, with appropriate adjustments for the effects of over-capacity conditions within a given segment. The method also meters demand to downstream segments when a segment's capacity is reached. The travel time results are aggregated to the facility level to produce the following performance measures for each analysis period:

- Average travel time
- Average speed
- Vehicle hours of delay

The PPEAG also describes how to estimate average density, level of service, and queuing at a section level once each section's demand-to-capacity ratio has been calculated.

The following steps are involved in performing a freeway facility analysis at a screening level:



Although it is feasible to perform a screening-level facility analysis by hand, it is more efficient to automate the process in a spreadsheet, such as the freeway planning tool developed for the PPEAG that was described in Section 11.2.5. The analysis process is as follows.

Step 1. Section the facility. Determine the start and end points of the facility and the number of 15-minute periods to include in the analysis are determined, following the guidelines given above. Next, divide the facility into sections, with section boundaries at each ramp gore point. Categorize each section as a basic, ramp, or weave section, following the guidance in Sections 11.3.2 through 11.3.4. Basic freeway segments should be split in two when a lane add or drop occurs, when a change in the terrain classification occurs, or when the free-flow speed changes as a result of changes in the roadway geometry or posted speed limit.

Step 2. Gather input data. All of the data required for a screening analysis of the individual freeway sections are required, plus the length of each section (see Section 11.3.1 for details).

Step 3. Determine section capacities. Determine the capacity of each freeway section by following the screening methods described in Sections 11.3.2 through 11.3.4.

Step 4. Determine 15-minute demand flow rates. This step is only required if 15-minute volumes are unavailable and is performed for the mainline volume entering the first section and for all ramp entering and exiting volumes. First, if hourly volumes are unavailable, convert each AADT to an hourly volume. The demand flow rate in each of the four 15-minute analysis periods is then synthesized using the following equation, derived from PPEAG Equation 17:

$$v_t = \begin{cases} V & t = 1, 3 \\ V \times \left(\frac{1}{PHF} \right) & t = 2 \\ V \times \left(2 - \frac{1}{PHF} \right) & t = 4 \end{cases}$$

where

v_t = demand flow rate during analysis period t (veh/h),

V = hourly volume (veh/h), and

PHF = peak hour factor (decimal).

This process assigns the highest volume to the second analysis period and the lowest volume to the last analysis period, while the first and third analysis periods are assigned flow rates equivalent to the average volume during the peak hour.

The mainline demand entering the first section during analysis period 2 should be compared to the section capacity. If the demand exceeds capacity, the study area should be expanded upstream.

The demand on each on-ramp during each analysis period should be compared to a capacity value of 2,000 veh/h/ln. If demand exceeds capacity, the excess demand is stored and added to the on-ramp demand for the following analysis period (note that this condition may mean that queues will spill back into the ramp terminal intersection and beyond).

The demand during analysis period 2 on each off-ramp during each analysis period should also be compared to a capacity value of 2,000 veh/h/ln. If demand exceeds capacity, queues will spill back from the ramp onto the mainline and the screening procedure is not applicable. If this situation happens, the analysis stops at this point and a more detailed analysis method should be considered.

Step 5. Assign demands to freeway sections. This process starts with the first freeway section during the first analysis period and works downstream to the end of the facility. It then repeats for each of the remaining analysis periods.

The entry demand to a given section consists of the mainline entering demand plus the on-ramp demand (if an on-ramp exists in the section), plus any unserved demand in the section carried over from the previous analysis period. This demand is compared to the section's capacity as follows:

- If demand is less than or equal to capacity, the demand for the off-ramp (if present) is used as-is. The mainline exiting demand is then the entry demand minus any off-ramp demand; this demand becomes the mainline entering demand in the next downstream section.
- If demand is greater than capacity, the section's entry volume is set to capacity and the excess demand is carried over to the next analysis period. The off-ramp volume is reduced in the same proportion as the mainline segment volume. The mainline exiting demand is then the section capacity minus the adjusted off-ramp demand; this demand becomes the mainline entering demand in the next downstream section.

Step 6. Calculate section d/c ratios. These are calculated for each section for each analysis period. The d/c ratio is the section entry demand divided by the section capacity.

Step 7. Estimate section travel times. The section travel time is estimated from three components: (1) the time to travel the section at the free-flow speed, (2) extra delay occurring during undersaturated (under-capacity) conditions, and (3) extra delay occurring as a result of oversaturated (over-capacity) conditions. The following equations are used, derived from Equations 20 through 22 in the PPEAG:

$$T_{i,t} = \frac{3,600L_i}{FFS_i} + L_i (\Delta_{RU_{i,t}} + \Delta_{RO_{i,t}})$$

with

$$\Delta_{RU_{i,t}} = \begin{cases} 0 & X_{i,t} < E \\ A(X_{i,t})^3 + B(X_{i,t})^2 + C(X_{i,t}) + D & E \leq X_{i,t} \end{cases}$$

$$X_{i,t} = \min(1, d_{i,t}/c_i)$$

$$\Delta_{RO_{i,t}} = \frac{900}{2L_i} (\max[1, d_{i,t}/c_i] - 1)$$

where

$T_{i,t}$ = travel time for section i during analysis period t (s),

L_i = length of section i (mi);

FFS_i = free-flow speed of section i (mph),

$\Delta_{RU_{i,t}}$ = undersaturated delay rate for section i during analysis period t (s/mi),

$\Delta RU_{i,t}$ = oversaturated delay rate for section i during analysis period t (s/mi),

A, B, C, D, E = parameters from Exhibit 11-12,

$d_{i,t}$ = demand in section i during analysis period t (veh/h),

c_i = capacity of section i (veh/h), and

900 = analysis period length (s) = 15 minutes.

Exhibit 11-12 Parameters for Screening-Level Freeway Speed Estimation Equation

FFS (mph)	A	B	C	D	E
75	68.99	-77.97	34.04	-5.82	0.44
70	71.24	-85.48	35.58	-5.44	0.52
65	92.45	-127.33	56.34	-8.00	0.62
60	121.35	-184.84	83.21	-9.33	0.72
55	156.43	-248.99	99.20	-0.12	0.82

Step 8. Estimate section and facility speeds. The average speed $S_{i,t}$ in each section i during analysis period t is calculated as follows:

$$S_{i,t} = \frac{3,600L_i}{T_{i,t}}$$

where all variables are as defined previously. The average facility speed $S_{F,t}$ during analysis period t is then:

$$S_{F,t} = \frac{3,600 \sum_i L_i}{\sum_i T_{i,t}}$$

Step 9. Estimate vehicle-hours of delay. The vehicle-hours of delay $VHD_{i,t}$ in section i during analysis period t , based on a threshold speed S_{TH} (mph) when delay is considered to begin, is calculated as follows:

$$VHD_{i,t} = \max \left(0, \frac{0.25v_{i,t} \times L_i}{S_{i,t}} - \frac{0.25v_{i,t} \times L_i}{S_{TH}} \right)$$

where all variables are as defined previously. The total facility vehicle-hours of delay is then simply the sum of all section VHD values across all analysis periods. The threshold speed is normally set as the posted speed for automobiles.

Detailed Analysis Method

Definition of a Freeway Facility

Chapter 10 of the HCM provides methods for evaluating the performance of freeway facilities. A facility consists of multiple contiguous freeway segments, extending up to

the distance that an automobile can drive at the posted speed in 15 minutes—typically 9 to 12 miles in urban areas and up to 15 miles in rural areas. In addition to the maximum length criterion, other criteria to be considered when defining the endpoints of a facility include freeway-to-freeway interchanges, urbanized area boundaries, major intersecting routes, major trip generators (e.g., central city downtowns, major airports), and the state border. It is important that the first and last segments of the facility operate below capacity throughout the defined analysis period, so that all the effects of any congestion that may occur are captured by the analysis.

For longer facilities, the analyst needs to carefully consider the demand inputs and the interpretation of results. The method assumes 15-minute demands to be applied instantaneously across the entire facility, even if travel times are longer than 15-minutes. In these cases, the delay, congestion, and queuing impacts are valid, but the temporal onset of congestion may be estimated to occur too early.

Study Period Length

The method accommodates study periods of 1 hour and longer, with an upper limit being set by (1) the needs of the analysis and (2) the capabilities of the software being used to apply the method. At a minimum, the entire facility should operate below capacity during the first and last 15-minute analysis periods. The FREEVAL software allows for up to 96 15-minute analysis periods to be analyzed, resulting in a full 24-hour study period.

Applicability

The method can be applied directly to general-purpose freeway lanes. Some aspects of the method (e.g., free-flow speed estimation) are also used as part of the analysis of merge, diverge, and weaving segments. Chapter 10 of the HCM provides extensions to the method for evaluating managed lane facilities and freeway work zones.

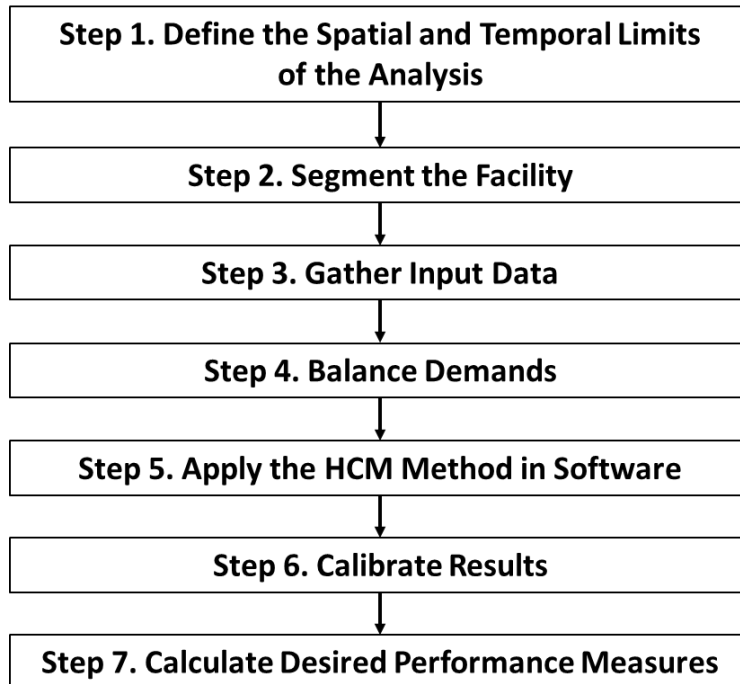
All of the detailed method limitations described in Sections 11.3.2 through 11.3.4 also apply to freeway facilities. In particular, if the capacity of an off-ramp roadway or terminal is exceeded, causing queues to spill back onto the mainline, the facility method cannot be used, and an alternative analysis tool will be required. The method does not directly estimate changes in demand resulting from congestion (e.g., motorists choosing alternative routes or modes), but the analyst can manually provide estimates of these changes with a demand adjustment factor. A demand adjustment factor (DAF) is a factor that is used to multiply all entering and/or exiting demands on the facility. The DAF can be less than 1.0 to estimate diversion effects to alternate facilities, or greater than 1.0 to estimate impacts of diversion onto the subject facility, or to evaluate future traffic growth scenarios.



Because detailed analysis is intended for near-term situations when all or nearly all inputs are known, and because CAVs are not yet commercially available, capacity adjustments for CAVs should not be made in detailed analyses.

Calculation Process

The following steps are involved in performing a freeway facility analysis at a detailed level:



The method estimates the average travel time required to traverse each freeway segment during each 15-minute analysis period, with appropriate adjustments for the effects of upstream and downstream bottlenecks (e.g., queue spillback, demand metering). The results are then aggregated to the facility level to produce the following performance measures for each analysis period:

- Average travel time
- Average speed
- Average density
- Vehicle miles traveled (VMT)
- Vehicle hours traveled (VHT)
- Vehicle hours of delay (VHD)
- Level of service

Since the freeway facility analysis captures multiple segments over multiple time periods, the results may be displayed in the form of contour plots or heat maps. The example in Exhibit 11-13 below shows a heat map of facility speeds, with each cell representing the result of a 15-minute time period for one segment. Cells are formatted to show high (free-

flow) speeds in green and slow (congested) speeds in red. The analysis shows is for a 34-segment facility that was analyzed over seven hours (28 time periods) from 2pm (14:00) to 9pm (21:00).

Exhibit 11-13 Speed Contour Example for Freeway Facilities Method

	Seg. 1	Seg. 2	Seg. 3	Seg. 4	Seg. 5	Seg. 6	Seg. 7	Seg. 8	Seg. 9	Seg. 10	Seg. 11	Seg. 12	Seg. 13	Seg. 14	Seg. 15	Seg. 16	Seg. 17	Seg. 18	Seg. 19	Seg. 20	Seg. 21	Seg. 22	Seg. 23	Seg. 24	Seg. 25	Seg. 26	Seg. 27	Seg. 28	Seg. 29	Seg. 30	Seg. 31	Seg. 32	Seg. 33	Seg. 34		
Analysis Period																																				
#1 14:00 - 14:15	70	65	65	69	63	63	63	68	69	63	70	63	69	63	70	65	70	70	70	69	70	65	69	69	70	65	65	70	69	70	63	62	61	62		
#2 14:15 - 14:30	69	65	65	69	63	63	63	68	69	63	70	63	69	63	70	65	69	70	70	68	70	68	70	65	69	68	70	65	65	70	68	70	63	60	60	60
#3 14:30 - 14:45	69	65	65	69	63	62	62	68	69	63	70	63	68	63	70	65	69	70	70	68	70	68	70	65	69	68	70	65	65	70	68	70	63	57	60	57
#4 14:45 - 15:00	69	65	65	69	62	62	62	68	69	63	70	63	68	63	70	65	69	70	70	68	70	68	70	65	69	68	70	65	64	70	68	70	62	55	59	55
#5 15:00 - 15:15	68	65	65	69	62	62	62	68	69	62	70	62	68	62	70	64	69	70	70	68	70	68	70	64	69	68	70	64	64	70	68	70	38	58	53	58
#6 15:15 - 15:30	67	65	65	69	62	62	62	68	69	62	70	62	68	62	70	64	69	70	70	68	70	68	70	64	69	68	70	64	64	70	68	70	21	58	48	58
#7 15:30 - 15:45	66	65	65	68	62	61	61	67	69	62	70	62	68	62	70	64	69	70	70	68	70	68	70	64	69	68	70	64	64	70	68	70	13	58	43	58
#8 15:45 - 16:00	66	65	65	68	62	61	61	67	69	61	70	61	68	61	70	64	69	70	70	68	70	68	70	64	69	68	70	64	64	69	68	70	8	58	39	58
#9 16:00 - 16:15	65	65	65	67	61	61	61	67	69	61	70	61	68	61	70	64	69	70	70	68	70	68	70	64	69	68	70	64	64	57	33	14	7	58	35	58
#10 16:15 - 16:30	64	64	65	67	61	61	61	67	69	61	70	61	68	61	70	64	69	70	70	68	70	68	70	64	69	46	27	20	17	20	14	12	7	58	32	58
#11 16:30 - 16:45	64	64	65	66	61	61	61	67	69	61	70	61	68	61	70	64	69	70	48	33	19	11	9	11	10	11	13	21	15	15	7	58	29	58		
#12 16:45 - 17:00	64	64	65	66	61	60	60	67	69	61	70	61	68	61	55	32	23	11	8	9	9	8	9	10	13	12	13	15	21	15	15	7	58	27	58	
#13 17:00 - 17:15	64	64	65	66	61	60	60	67	69	61	69	37	28	12	9	9	8	7	7	11	9	9	10	13	12	13	15	22	15	15	7	58	25	58		
#14 17:15 - 17:30	66	65	65	67	62	61	61	66	52	16	10	7	11	10	10	12	10	9	10	14	10	11	11	16	12	11	15	22	15	15	7	58	24	58		
#15 17:30 - 17:45	67	65	65	69	54	54	44	44	21	12	13	9	12	10	11	12	8	7	7	10	10	9	10	15	12	13	15	21	16	15	7	58	24	58		
#16 17:45 - 18:00	68	65	65	68	45	35	20	20	12	9	11	8	12	9	11	12	11	11	8	11	10	10	10	11	14	12	13	15	21	16	15	7	58	25	58	
#17 18:00 - 18:15	69	65	65	68	49	43	20	20	14	10	12	9	12	10	10	10	7	8	8	14	11	10	11	15	12	12	15	22	16	15	7	58	26	58		
#18 18:15 - 18:30	70	65	65	69	63	63	51	51	18	9	10	7	11	11	10	11	9	8	8	12	10	11	12	16	13	14	16	21	17	15	7	58	27	58		
#19 18:30 - 18:45	70	65	65	69	63	63	63	68	69	16	11	9	12	11	13	13	11	9	8	14	11	11	11	15	13	15	16	22	16	15	7	58	29	58		
#20 18:45 - 19:00	70	65	65	69	64	64	64	68	70	56	41	39	46	15	11	11	11	10	9	13	12	10	12	16	14	13	17	22	16	15	7	58	32	58		
#21 19:00 - 19:15	70	65	65	69	64	64	64	68	70	65	70	65	69	58	37	23	12	10	9	15	12	11	11	16	15	16	13	21	16	16	7	58	37	58		
#22 19:15 - 19:30	70	65	65	69	64	64	64	68	70	65	70	65	69	65	70	66	70	63	31	24	19	13	12	17	14	13	14	22	17	16	7	58	43	58		
#23 19:30 - 19:45	70	65	65	69	64	64	64	68	70	65	70	65	69	65	70	66	70	70	70	69	70	66	69	43	30	26	21	25	16	16	7	58	51	58		
#24 19:45 - 20:00	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	37	28	7	58	65	58		
#25 20:00 - 20:15	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	18	58	70	58		
#26 20:15 - 20:30	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	68	70	67	70		
#27 20:30 - 20:45	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	66	70	65	70		
#28 20:45 - 21:00	70	65	65	69	64	64	64	68	70	66	70	66	69	66	70	66	70	70	70	69	70	66	69	69	70	66	66	70	69	70	66	70	65	70		

The speed contours show several hours of congestion from roughly 3:30pm to 8pm, and a queue that extends at its maximum to segment 5. The boundaries of the analysis (first and last time period and first and last segment) are not congested, suggesting a well-defined analysis period. For a freeway facility analysis, all congestion should preferably be fully contained within the specified time-space domain.

Due to the number of calculations involved, software is necessary to perform a detailed freeway facilities analysis. Analysis tools that were available at the time of writing for performing a freeway facilities analysis were described in Section 11.2.5. The analysis process is as follows.

Step 1. Define the spatial and temporal limits of the analysis. The start and end points of the facility and the number of 15-minute periods to include in the analysis are determined, following the guidelines given above.

Step 2. Segment the facility. The facility is first divided into sections between ramp gore points. Next, merge, diverge, and weave segments are identified, following the guidance in Sections 11.3.2 and 11.3.3. The remaining unassigned portions of the facility then become basic freeway segments. Basic freeway segments should be split when a lane add or drop occurs, when a change in the terrain classification occurs, or when the free-flow speed changes as a result of changes in the roadway geometry.

Some adjustments to segment boundaries may be required in the case of closely spaced ramps (where ramp influence areas overlap each other) and very long weaving segments (where the segment length exceeds the maximum weaving length). Step A-2 of the HCM methodology, starting on page 10-24 of the HCM 7th Edition, details the segmentation process, including these special cases.

Step 3. Gather input data. All of the data required for a detailed analysis of the individual freeway segments are required, except for the peak hour factor (see Section 11.3.1 for details). In lieu of a peak hour factor, 15-minute volumes must be provided for each segment for each analysis period.

The length of each segment is also required, along with two parameters not used by segment-based analyses: jam density and queue discharge capacity drop. These parameters can initially be defaulted to 190 pc/h/ln and 7%, respectively, if local values are not available. Appendix 11B describes how these parameters can be used later in the process to calibrate the method to match field conditions.

Step 4. Balance demands. Both the total entering and the total exiting demand for the facility (considering both the freeway mainline and all on- and off-ramps located along the facility) should be determined for each analysis period. If the entering demand does not equal the exiting demand (for example, because traffic counts are used to estimate demand and congestion is occurring along the facility that prevents entering demand from reaching its desired exit), the demands in that time period will need to be adjusted. Step A-4 of the HCM methodology, starting on page 10-28 of the HCM 7th Edition, describes the balancing process.

Step 5. Apply the HCM methodology in software. All of the information about the facility is coded into the software and the software is used to determine individual segment capacities, v/c ratios, and locations and extent of congestion.

Step 6. Calibrate results. The initial software results should be compared to available information about existing conditions. If necessary, stepwise adjustments can be made first to free-flow speed, then capacity, and (as a last resort) to demand to match existing conditions. Appendix 11B describes this process.

Step 7. Calculate desired performance measures. Once the facility has been satisfactorily calibrated, the software can then be used to report performance measures for both existing conditions and desired future scenarios that the analyst subsequently codes. Vehicle-hours of delay are calculated by comparing actual travel times to the travel time at the posted speed limit. The portion of the travel time greater than the travel time at the speed limit is considered to be delay.

11.3.5 Multilane Highway Facilities



No guidance is presented for the effects of CAVs on multilane highway capacity because no research has been conducted yet for these roadway types. However, it may be reasonable to apply the capacity adjustments for basic freeway segments provided in Appendix 6B to multilane highways for broad-brush and screening level analyses, given the similar operating characteristics and methodologies for freeways and multilane highways.

Broad-Brush Analysis Method

Exhibit 11-14 contains a table providing estimates of design-hour, peak-direction capacities along multilane highways without traffic signals. It is applied in the same way that Exhibit 11-11 in Section 11.3.4 is applied.

Exhibit 11-14 Generalized Multilane Highway Capacities (veh/h)

Area Type	Terrain	Posted Automobile Speed Limit (mph)				
		45	50	55	60	65
Urban	Level	3,620	3,800	3,980	4,160	4,345
	Rolling	3,455	3,625	3,800	3,975	4,145
	Mountainous	3,165	3,325	3,485	3,640	3,800
Rural	Level	2,815	2,955	3,100	3,240	3,380
	Rolling	2,345	2,465	2,580	2,700	2,815
	Mountainous	1,760	1,850	1,935	2,025	2,110

Note: Assumptions used in this table are: HV% = 5 (urban), 25 (rural); PHF = 0.95 (urban), 0.88 (rural); 2 lanes per direction; free-flow speed = posted speed + 5 mph; driver population familiar with the facility.

Example 11-11 Multilane Highway Capacity Analysis (Broad-Brush Method)

A rural four-lane multilane highway (two lanes in each direction) is located in rolling terrain with a 55-mph speed limit. The AADT is 21,700, the K factor is 16.2, the D factor is 62, and the heavy-vehicle percentage is 18.6.

The design-hour volume V is:

$$V = AADT \times \frac{K}{100} \times \frac{D}{100} = 21,700 \times \frac{16.2}{100} \times \frac{62}{100} = 2,180 \text{ veh/h}$$

The capacity obtained from Exhibit 11-14, which assumes 25% heavy vehicles, is 2,580 veh/h. An adjusted local capacity can be determined as follows by substituting the local heavy-vehicle percentage and keeping the table values for all other inputs that are unknown or unchanged:

$$c_{adj} = c_{table} \times \frac{PHF_{local}}{PHF_{table}} \times \frac{1 + (E_T - 1)(\%HV_{table}/100)}{1 + (E_T - 1)(\%HV_{local}/100)} \times \frac{N_{local}}{2} \times CAF_{pop}$$

$$c_{adj} = 2,580 \times \frac{0.88}{0.88} \times \frac{1 + (3 - 1)(25/100)}{1 + (3 - 1)(18.6/100)} \times \frac{2}{2} \times 1.00 = 2,820 \text{ veh/h}$$

The v/c ratio is then $(2,180 / 2,820) = 0.77$.

Screening Analysis Method

Definition of a Multilane Highway Facility

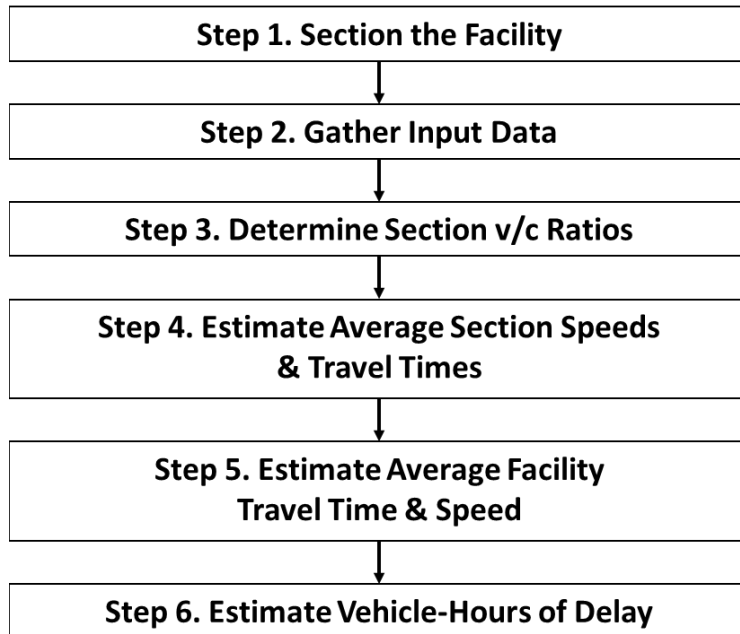
Section I6 of the *Planning and Preliminary Engineering Applications Guide to the HCM* (PPEAG) provides a method for evaluating the performance of a multilane highway facility. A facility consists of one or more contiguous multilane highway sections, along with the signalized intersection (if present) at the end of the facility. The maximum facility length should be the distance that an automobile can drive at the posted speed in 15 minutes. A facility boundary should also be established at a location where the highway transitions to a different facility type (e.g., two-lane highway, freeway, urban street). In addition to these criteria, other criteria that could be considered when establishing the endpoints of a facility include urbanized area boundaries and major intersecting routes.

Applicability

The method can be used to evaluate highway performance during the design hour on multilane highway facilities with free-flow speeds between 45 and 70 mph.

Calculation Process

The process described below estimates average travel time and average speed by section and facility and vehicle-hours of delay for the facility as a whole. The PPEAG also describes how to calculate average density and level of service for multilane highways. The following steps are involved:



Step 1. Section the facility. Determine the start and end points of the facility, following the guidance above. Next, divide the facility into one or more basic sections. Basic section boundaries should be established at signalized intersections, major unsignalized intersections where highway demand changes significantly, lane adds or drops, locations where a change in the terrain classification occurs, and locations where the free-flow speed changes as a result of changes in the roadway geometry or posted speed limit. If interchanges are located along the highway, ramp or weaving sections (as appropriate) will also need to be established, following the guidance for freeways in Section 11.3.5.

Step 2. Gather input data. All of the data required for a screening analysis of a basic multilane highway sections are required, plus the length of each section (see Section 11.3.1 for details). In addition, if the facility ends at a signalized intersection, the average control delay for the highway mainline is a required input. Refer to Chapter 13 for procedures to calculate control delay at signalized intersections.

Step 3. Determine section v/c ratios. Determine the v/c ratio for each multilane highway section by following the screening methods described in Sections 11.3.1 through 11.3.3.

Step 4. Estimate average section speeds and travel times. The following equation, derived from PPEAG Equation 40, is used to estimate average speeds (not including any intersection delay) along a multilane highway section:

$$S_{u,i} = \frac{FFS_i}{1 + a \times (v_i/c_i)^b}$$

where

$S_{u,i}$ = uninterrupted-flow average travel speed for section i (mph),

FFS_i = free-flow speed of section i (mph),

a, b = parameters from Exhibit 11-15,

v_i = demand volume in section i (veh/h), and

c_i = capacity of section i (veh/h).

Exhibit 11-15 Parameters for Multilane Highway Speed Estimation

Free-Flow Speed (mph)	a	b
70	0.37	6.9
65	0.27	7.3
60	0.23	7.5
55	0.18	7.7
50	0.13	8.1
45	0.07	8.9

The average travel time for a multilane highway section, including control delay at a signalized intersection (if any) at the end of the section is then:

$$TT_i = \frac{3,600L_i}{S_{u,i}} + d_i$$

where

TT_i = average travel time in section i (s),

L_i = length of section i (mi),

S_i = uninterrupted-flow average travel speed for section i (mph), and

d_i = average control delay at the downstream signal (s).

The average travel speed S_i for section i , including any signalized intersection delay, is then (derived from PPEAG Equation 42):

$$S_i = \frac{3,600L_i}{TT_i}$$

where all variables are as defined previously.

Step 5. Estimate average facility travel time and speed. The average facility travel time TT_F (in seconds) is the sum of the individual section travel times, while the average facility speed S_F (in mph) is (PPEAG Equation 42):

$$S_F = \frac{3,600L}{TT_F}$$

where L is the facility length in miles.

Step 6. Estimate vehicle-hours of delay. The vehicle-hours of delay VHD_i for section i , based on a threshold speed S_{TH} (mph) when delay is considered to begin, is calculated as follows:

$$VHD_i = \max\left(0, \frac{v_i \times L_i}{S_i} - \frac{v_i \times L_i}{S_{TH}}\right)$$

where all variables are as defined previously. The total facility vehicle-hours of delay is then the sum of all section VHD values. The threshold speed is normally set as the posted speed for automobiles.

Detailed Analysis Method

The HCM does not provide a detailed method for evaluating multilane highway facility performance. Capacity problems usually arise at signalized intersections along the highway and not along basic segments. Other performance measures can be estimated using the screening method described above or by using alternative analysis tools.

11.4 Two-Lane Highways



No guidance is presented for the effects of CAVs on two-lane highway capacity because no research has been conducted yet for these roadway types.

Two-lane highway operations are characterized by passing maneuvers, formation of platoons within the traffic stream, and delay experienced by trailing (follower) vehicles while unable to pass lead vehicles. As opposing and passing demand increases, the ability to pass slower vehicles decreases as the number of followers increase due to limited passing opportunities. Quality of service becomes eventually unacceptable even for lower volume-to-capacity (v/c) ratios. Hence, the use of volume-to-capacity ratio by itself is not a good performance measure for two-lane highway analysis. The v/c ratio calculation for two lane highways is very basic as it is just the 15-minute demand flow rate divided by a fixed capacity value. This can create a frequently misleading underestimated operational indication as it does not reflect any of the driver behavior present (i.e. platooning, inability to maintain desired speed, etc.) on a two-lane highway.

Different performance measures are required to properly reflect two-lane highway operations. This includes free-flow speed, average speed, percent followers and follower density. Percent followers are estimated by determining the number of vehicles in a “following” state based on a headway threshold (e.g. 2.5s in HCM 7 and 3.0s for APM methods). Higher percentages of followers indicate the potential of longer platoons and inadequate passing opportunities (e.g. lack of passing lanes or long enough zones). Follower density is the number of followers per mile per lane. Follower density generally increases with the directional flow rate. Like other density-based measures, the follower density has been equated to LOS ranges for the ease of reporting for both the APM and HCM methods.



For the purposes of reporting, the volume-to-capacity measure should still be shown with the caveats noted in this section for consistency with established mobility targets and design guidelines. The v/c ratio shall be supplemented by, as applicable, speeds, percent followers, follower density, and LOS performance measures. of highway performance.

When computing v/c ratios, the capacity of a directional two-lane highway segment under base conditions (no passing lane) is 1,700 vehicles per hour per lane (vphpl), with a limit of 3,200 vph for both directions. This can be seen in HCM Exhibit 15-5 where the maximum passing lane capacity is limited to 1,500 vphpl when combined with the maximum opposing (no-passing) directional capacity of 1,700 vphpl. Divide the directional 15-min flow rate (vph) by the appropriate capacity value to obtain the reportable segment v/c.

Estimation of capacity may still be needed for certain conditions such as for evacuation planning, special event planning, and evaluation of the downstream impacts of incident bottlenecks once cleared. However, use the capacity estimation for judging events, not for a volume-capacity calculation. For a complete description of the capacity estimation and the details of the two-lane highway methodology, refer to Chapter 15 of the HCM.

11.4.1 Analysis Segment Classification

Two-lane highway analysis requires determination of the appropriate methodology to apply. This is based on federal functional classification, area type or land use context. Area type or land use context is used to determine whether the APM or HCM analysis methodology applies. The federal functional class (i.e. rural minor arterial, rural major collector, etc.) is used to determine which classification under the APM methodology applies. Two-lane highways are classified into Class I, Class II, or Class III analysis designations:

Class I (APM methodology): These are rural 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 connections between facilities that serve long-distance trips. Rural Principal Arterials mostly act as Class I highways. Coos Bay-Roseburg Highway-OR 42 (Hwy # 035) is an example of a Class I highway.

Class II (APM methodology): These are rural highways where motorists do not necessarily expect to always 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. Rural Minor Arterials and Rural Major Collectors mostly act as Class II highways. For instance, West Diamond Lake Hwy- OR 230 (Hwy # 233 that connects Crater Lake Hwy (OR 62) and Diamond Lake Hwy (OR 138) primarily serves recreational trips and passes through undeveloped, rugged terrain.

Class III (HCM methodology): These are special cases serving moderately developed areas. They may be portions of a Class I or Class II highway that pass through small towns, suburban areas, unincorporated communities, 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. These segments need to be two lanes, so any two-way left turn lane segments are not included and need to be used in the urban street methodology.

- Some example sections:
 - Surf Pines/Camp Rilea to Warrenton section on Oregon Coast Highway (US101; Hwy #009)
 - Detroit city section on N Santiam Hwy (OR22; Hwy #162)
 - Richland city section on Baker – Copperfield Highway (OR86 (Hwy #012)

The rural US 101 section from Surf Pines (north of Gearhart) to Warrenton is a spread-out recreational area with substantial development along the highway. The Detroit and Richland sections of the highways pass through small towns having speed restrictions, significant roadside developments and unsignalized access points.

11.4.2 Analysis Segmentation

Two-lane highway analysis is done by direction consistent with other HCM facility-based methodologies which allow compatibility with the multilane highway analysis methods that may come into play as a build alternative. An example of this would be long multi-mile passing lanes that function as a multilane highway section (see Appendix 10A)

instead of a typical shorter passing section. Two-lane highway analysis requires each direction to be broken into various analysis segments beyond the methodological-based ones described in the previous section.

Signalized intersections, all-way stops, or roundabouts must be spaced more than two miles apart for this methodology to be compatible. If major intersection spacing is less than two miles, then the urban street methodology (see HCM Chapters 16-18) or typical intersection-based or micro-simulation analysis tools (e.g. Synchro/SimTraffic) need to be used instead. Intersections that force mainline traffic to stop or yield create segment start or end points. Advanced analysis (e.g. SIDRA mesoscopic lane-based analysis or micro-simulation) may be necessary if the intersection approach exceeds capacity as the resulting queuing will impact the adjacent two-lane highway section upstream of the intersection. The two-lane analysis results for this section will not be accurate in this case.

Directional segments need to be identified through the study area. Each segment should be relatively homogeneous as much as possible. Lane striping will cause the directional segment breaks to be in different locations. The primary analysis segments should break the study section by:

- Functional class/area type/land use context
- Any intersection that requires mainline traffic to stop or yield.
- Passing-constrained (e.g. no passing based on striping)
- Passing zones (e.g. 100% passing based on striping)
- Passing/climbing lanes (do not count any slow-moving vehicle turnouts as a passing lane)

Once the primary segments are identified, then they should be further broken by:

- Posted speed limit changes (HCM methodology only)
- Changes to lane or shoulder widths (HCM methodology only)
- A change of more than 10% in directional hourly demand volume or heavy vehicle percentage at unsignalized two-way stop-controlled intersections where the analysis direction does not stop.
- Terrain (e.g. level, rolling, mountainous) changes
- Horizontal and vertical curves (HCM methodology only)

After all segment breaks have been identified, any segments less than 1320' (0.25 mile) should be combined with adjacent segments as the HCM methodology does not analyze shorter segments. This may not always be completely possible, especially in Class III

segments where there may be multiple changes (i.e. speed, volumes). Any segments of less than 1320' will be analyzed in HCS at 1320' however speeds and follower density will still be relative. Use of probe data-based free-flow speeds will remove lane and shoulder width changes from analysis for example.

11.4.3 Follower Density Models for Class I and Class II Highways

The Oregon Department of Transportation (ODOT) has conducted studies to develop alternative LOS criteria for two-lane highway analysis¹. These studies are available within a consolidated [supplemental materials](#) document on the APM website. The studies were based on the framework adopted for empirical investigation of two-lane rural highway performance indicators in Montana but using actual Oregon highway sections². The study uses follower density as a performance measure to describe two-lane highways operations. Follower density is the number of followers in a directional traffic stream over a unit length of a highway. Followers are vehicles travelling with headway less than 3.0 seconds (*HCM 2010 & 6th Edition*; 2.5s in *HCM 7th Edition*). The argument behind using this performance indicator is that a road with a lower average daily traffic (ADT) and high time spent following should have a lower LOS than the same road with a higher ADT and an equal time spent following³. Unlike other performance measures, follower density takes into consideration the effect of the traffic level on highway performance². Generally, density measures are difficult to directly measure in the field, but it can be estimated at point locations from volume and speed measurements from permanent or temporary traffic count detectors.

Similar to the HCM methodology, the ODOT study developed LOS criteria for Class I and II two-lane highways. The study developed relationships between follower density (veh/mile/lane) and platooning variables for the best statistical significance. Exhibit 11-17 lists the follower density models.

The platooning variables included in the follower density models are:

- Traffic flow in the direction of travel (vph),
- Opposing traffic flow (vph),
- Percent heavy vehicles (%) (FHWA Classes 6-13),
- Percent no-passing zones (%),

¹ Modeling Follower Density on Two-Lane Rural Highways; and Modeling Performance Indicators on Two-Lane Rural Highways: The Oregon Experience

² Al-Kaisy, A., and Karjala, S. (2008). Indicators of Performance on Two-Lane Rural Highways: Empirical Investigation, Transportation Research Record, No. 2071, pp. 87–97.

³ Van As, C. (2003). The Development of an Analysis Method for the Determination of Level of Service on Two-Lane Undivided Highways in South Africa. South African National Roads Agency, Pretoria.

- Rolling Terrain⁴ (1 = Rolling Terrain, 0 = Otherwise), and
- Mountainous Terrain⁴ (1 = Mountainous Terrain, 0 = Otherwise).

Exhibit 11-16 Follower Density Models by Two-Lane Highway Class

Functional Class	Model Form	R ²
Class I Highways	Follower Density = $-0.1917 + 0.005953 (\text{Traffic Volume}) + 0.0005167 (\text{Opposing Volume}) + 0.0006739 (\% \text{ Heavy Vehicles}) + 0.0002392 (\% \text{ No Passing}) + 0.05248 (\text{Rolling Terrain})$	0.81
Class II Highways	Follower Density = $-0.1784 + 0.006189 (\text{Traffic Volume}) - 0.0001607 (\text{Opposing Volume}) + 0.0006163 (\% \text{ Heavy Vehicles}) + 0.0006055 (\% \text{ No Passing}) + 0.0168 (\text{Rolling Terrain}) + 0.03994 (\text{Mountainous Terrain})$	0.75

Follower density acts as a surrogate measure to assess operations of rural two-lane highways. Example 11-12 and Example 11-13 outline the application of these procedures. Follower density is most significantly affected by traffic volume and opposing volume. Percent heavy vehicles, percent no-passing zones and terrain type have a much lesser effect. The effect of these variables on Class II highways is somewhat greater than on Class I highways. The overall effect of these variables will not affect LOS unless it is near a boundary condition. Percent heavy vehicles and terrain type are readily available. Percent no-passing zone data is typically collected from milepoint-based videolog data or aerial photography which may be somewhat time consuming to gather, and could be defaulted to a rough estimate such as 25%, 50%, 75% etc. The follower density thresholds are established at each LOS category as listed in Exhibit 11-18.

Exhibit 11-17 LOS Criteria by Two-Lane Highway Class

LOS	Class I Highways	Class II Highways
	Follower Density (veh/mile/lane)	Follower Density (veh/mile/lane)
A	≤ 2	≤ 2.5
B	$> 2 - 3.5$	$> 2.5 - 4.0$
C	$> 3.5 - 6.0$	$> 4.0 - 6.5$
D	$> 6.0 - 9.0$	$> 6.5 - 10.0$
E	> 9.0	> 10.0

At LOS A, operating speeds are close to the posted speeds with a low percent followers

⁴ Terrain is equated to: Level for grades less than 3% ; Rolling for grades between 3 to 6%; and Mountainous for grades greater than 6 %

which equates to little passing difficulty. At capacity (LOS E), operating speeds will likely still be close to posted, but platoons of followers indicating a high follower density will be evident and passing will be very difficult at best. Attempting to pass at these levels can lead to increased risk of head-on and sideswipe collisions. If demand exceeds capacity (adjusted directional flow rate divided by the 1700 vph maximum capacity) then the segment is at LOS F, and this method does not apply and requires advanced analysis tools such as micro-simulation. This d/c calculation should be done to check for the over-capacity condition (LOS F) as part of the overall analysis.

This LOS-based methodology should be used for the analysis of rural state two-lane highways and can be used for similar county and other jurisdiction roadways. This methodology can be used on a planning analysis basis for corridor plans and other high-level screening efforts using AADT, K30 and D30 factors from ATR's to develop 30th highest hour volumes as well as information from online databases. Using directional and classification tube counts, analysis can be created for more detailed refinement/facility plans and projects.

Class I and II sections with poor LOS may indicate that a slow-moving vehicle turnout, passing lane, climbing lane, or multilane section is needed. Since the HCM 7th edition follower definition is 2.5s vs 3.0s used in the ODOT studies, the APM methodology will be slightly more conservative than the HCM for these sections. The two methods have different inputs, but most importantly, the ODOT method is based on data from similar Oregon highways instead of national data in the HCM. The follower density may vary between the two methods and the LOS ranges are generally wider in the HCM, but the resulting LOS should not be more than a grade different. For passing, climbing, and multilane sections follow the HCM methodologies.

Class III sections with poor LOS may indicate that turn lanes, a two-way left turn median, or additional through lanes may be necessary. Adding these elements will convert a section to an urban street and should be analyzed using that HCM methodology and possibly using tools like SIDRA Intersection (see Appendix 12/13A) or microsimulation (see Chapter 15).

Example 11-12 Class I Highway LOS

This example demonstrates the application of follower density-based LOS criteria for Class I highway to find the expected LOS in each direction on the two-lane highway segment as described below:

Input Data

- Albany-Corvallis Highway (US20; Hwy #031), MP 6.41 – 7.41
- Peak hour volume = 1788 vph (both directions; 2017 last count)
- Directional split (during analysis period) = 49% EB and 51% WB
- PHF = 0.92
- 4 % trucks EB ; 7 % trucks WB
- 1-mile segment length
- 43% no-passing zones EB ; 72% no-passing zones WB
- Level terrain

LOS by Follower Density Model

This highway section is a rural principal arterial which links two major cities (Albany and Corvallis) and is an important commuter corridor. Therefore, this segment is categorized as Class I.

Follower density model:

Follower Density = $-0.1917 + 0.005953 (\text{Traffic Volume}) + 0.0005167 (\text{Opposing Volume}) + 0.0006739 (\% \text{ Heavy Vehicles}) + 0.0002392 (\% \text{ No Passing}) + 0.05248 (\text{Rolling Terrain})$

LOS criteria:

LOS	Class I Highways
	Follower Density (veh/mile/lane)
A	≤ 2.0
B	$> 2.0 - 3.5$
C	$> 3.5 - 6.0$
D	$> 6.0 - 9.0$
E	> 9.0

EB directional analysis:

Traffic flow rate = volume / PHF = $(1788 \times 0.49) / 0.92 = 952$ vph

Demand to capacity ratio = adjusted directional flow rate / 1700 vph = $952 / 1700 = 0.56$

The demand is less than capacity, so LOS F conditions are not present, and the rest of the analysis can be completed. This value could also be reported out and compared with applicable v/c targets; however it should be noted that this is a simplified secondary measure and will not necessarily match well with the LOS. In this case, the resulting v/c implies better conditions that are not realized than the final LOS results as seen below.

Opposing traffic flow rate = opposing volume / PHF = $(1788 \times 0.51) / 0.92 = 991$ vph

Percent Heavy Vehicles = 4 %

Percent No Passing zone = 43%

Rolling Terrain = 0 as terrain is considered "Level"

Follower Density = $-0.1917 + 0.005953 (952) + 0.0005167 (991) + 0.0006739 (4) + 0.0002392 (43) + 0.05248 (0)$
= 6.0 veh/mile/lane

LOS is C-D for the analysis direction.

WB directional analysis:

Traffic flow rate = volume / PHF = $(1788 \times 0.51) / 0.92 = 991$ vph

Demand to capacity ratio = adjusted directional flow rate / 1700 vph = $991 / 1700 = 0.58$

The demand is less than capacity, so LOS F conditions are not present, and the rest of the analysis can be completed.

Opposing traffic flow rate = opposing volume / PHF = $(1788 \times 0.49) / 0.92 = 952$ vph

Percent Heavy Vehicles = 7 %

Percent No Passing zone = 72 %

Rolling Terrain = 0 as terrain is considered "Level"

Follower Density = $-0.1917 + 0.005953 (991) + 0.0005167 (952) + 0.0006739 (7) + 0.0002392 (72) + 0.05248 (0)$
= 6.4 veh/mile/lane

LOS is D for the analysis direction.

Example 11-13 Class II Highway LOS

This example demonstrates the application of follower density-based LOS criteria for a Class II highway to find the expected LOS in each direction on the two-lane highway segment.

Input Data

- West Diamond Lake Hwy (OR230; Hwy #233), MP 5.86 -6.86
- Peak hour volume = 109 vph (total in both directions; 2025 data)
- Directional split (during analysis period) = 54 % NB and 46 % SB
- PHF = 0.80
- 31 % trucks NB; 30 % trucks SB
- 1-mile segment length
- 0% no-passing zones NB & SB
- Rolling terrain

LOS by Follower Density Model

This highway section is a rural minor arterial which serves primarily scenic and recreational destinations (i.e. Crater Lake National Park), passes through rugged terrain, and high travel speeds are not expected in all places. This highway is good example of Class II.

Follower density model:

Follower Density = $-0.1784 + 0.006189 (\text{Traffic Volume}) - 0.0001607 (\text{Opposing Volume}) + 0.0006163 (\% \text{Heavy Vehicles}) + 0.0006055 (\% \text{No Passing}) + 0.0168 (\text{Rolling Terrain}) + 0.03994 (\text{Mountainous Terrain})$

LOS criteria:

LOS	Class II Highways
	Follower Density (veh/mile/lane)
A	≤ 2.5
B	$> 2.5 - 4.0$
C	$> 4.0 - 6.5$
D	$> 6.5 - 10.0$
E	> 10

Analysis on 53 % direction of flow (NB)

Traffic flow rate = volume / PHF = $(109 \times 0.54) / 0.80 = 74$ vph

Opposing traffic flow rate = opposing volume / PHF = $(109 \times 0.46) / 0.80 = 63$ vph

Demand to capacity ratio = adjusted directional flow rate / 1700 vph = $74 / 1700 = 0.04$

The demand is less than capacity, so LOS F conditions are not present, and the rest of the analysis can be completed.

Percent Heavy Vehicles = 31 %

Percent No Passing zone = 0%

Rolling Terrain = 1 as terrain is considered “Rolling”

Mountainous Terrain = 0 as terrain is considered “Rolling” type

Follower Density = $-0.1784 + 0.006189 (74) - 0.0001607 (63) + 0.0006163 (31)$
 $+ 0.0006055 (0) + 0.0168 (1) + 0.03994 (0)$

= 0.31 veh/mile/lane

LOS is A for the analysis direction.

Analysis on 47 % direction of flow (SB)

Traffic flow rate = volume / PHF = $(109 \times 0.46) / 0.80 = 63$ vph

Demand to capacity ratio = adjusted directional flow rate / 1700 vph = $63 / 1700 = 0.04$

The demand is less than capacity, so LOS F conditions are not present, and the rest of the analysis can be completed.

Opposing traffic flow rate = opposing volume / PHF = $(109 \times 0.54) / 0.80 = 74$ vph

Percent Heavy Vehicles = 30 %

Percent No Passing zone = 0%

Rolling Terrain = 1 as terrain is considered “Rolling”

Mountainous Terrain = 0 as terrain is considered “Rolling”

Follower Density = $-0.1784 + 0.006189 (63) - 0.0001607 (74) + 0.0006163 (30)$
 $+ 0.0006055 (0) + 0.0168 (1) + 0.03994 (0)$

= 0.22 veh/mile/lane

LOS is A for the analysis direction.

11.4.4 Class III Highways Methodology

Oregon-based preliminary models developed for Class III highways were limited because of limited sample size. Class III segments should use the current HCM methodology and related LOS criteria. Motorists are expected to travel at or near the posted speed limit on these facilities. The LOS criteria for two-lane Class III highways are shown in Exhibit

11-19. Note that since speeds are assumed to be lower, there is a higher follower density tolerance compared to Exhibit 11-18.

For a complete description of the methodology, refer to Chapter 15 of the HCM. Class III highways in this chapter are referred to as “Lower Speed Highways” in the HCM where the posted speed limit is less than 50 mph. However, the following steps provide a summary of the HCM methodology with additional guidance on steps or parameters that need to be calculated for starting the analysis. It is assumed that HCS (Highway Capacity Software) using the “Two-Lane Facility” module is used to facilitate the analysis as that would likely be the most likely tool to be used as manual computations are complex and time consuming.

Exhibit 11-18 Class III/Lower Speed Highways LOS Criteria

LOS	Class III Highways ¹
	Follower Density (veh/mile/lane)
A	≤ 2.5
B	$> 2.5 - 5.0$
C	$> 5.0 - 10.0$
D	$> 10.0 - 15.0$
E	> 15.0

¹From HCM 7th Edition, Exhibit 15-6. “Lower Speed Highways”

Step 1: Facility Segmentation

Following the guidance in Section 11.4.1 break the Class III segment into homogeneous sections. Most likely these will be passing-constrained sections with limited or no passing as longer passing zones or passing lanes would not be expected in a built-up/dense area or passing would not be possible due to roadway geometry, for example, in a rural recreational area. Any location where the mainline must stop or yield creates a section break. Further break the segment for changes in lane or shoulder width (note that 6' or wider shoulders are limited to 6' so many shoulder width changes can be ignored), speed limit, horizontal curvature, vertical terrain, or significant change in numbers of access points.

Refer to HCM Exhibit 15-10 to verify that the above segments fall into the acceptable length ranges for roadway passing type and vertical terrain (e.g. 0.25 to 3.0 miles). Make sure all the required data can be obtained for each segment. For these kinds of communities, it is likely that necessary segments will be less than the 0.25 minimum segment length because of shoulder or speed changes, so warnings from HCS can be ignored. For state highways, use the [Horizontal Grade Report](#) and the [Vertical Grade Report](#) to help determine the segmentation (see HCM's Step 3 for vertical and Step 5 for horizontal guidance). Use of software such as Highway Capacity Software (HCS) will streamline the segmentation, data organization, and the following analysis steps.

Step 2: Determine demand flow rates and d/c ratio check

Calculate the directional demand flow rate in the analysis direction and the opposing direction with the matching peak hour factors (PHF) using HCM Equation 15-1.

$$v_i = \frac{V_i}{PHF}$$

where

- v_i = directional demand flow rate i (vph);
- i = “d” (analysis direction) or “o” (opposing direction);
- V_i = demand volume for direction i (vph);

Determine the d/c ratio for the segment by dividing v_i by 1700 vph (maximum directional capacity). If the resulting d/c ratio is greater than 1.00 then the section is overcapacity and LOS F conditions exist. The rest of the steps are skipped. Further analysis could likely involve micro-simulation, intersection analysis, or the urban street methodology (HCM Chapter 16).

Step 3: Determine vertical terrain class

Determine the percent grade for each segment for state highways using the [Vertical Grade Report](#). This value along with the segment length would be entered into HCS to determine the vertical terrain class (HCM Exhibit 15-11).

Step 4: Estimation of Free Flow Speed (FFS)

Ideally, FFS would be directly field measured or, more likely, can be historically estimated by using the average speed using an overnight or other low demand hour (e.g. under 200 vph) using the Performance Chart tool in [RITIS](#). RITIS input parameters should be consistent with the counts used (i.e. same year, 24-hour duration, at least the same month or same season, weekdays, etc.), so that the speeds are consistent to minimize variation.

However, if software like HCS is used as would normally be expected, the measured FFS has a minimum value of 45 mph. If the Class III segment has a RITIS FFS of 45 mph or greater than the “Measured” FFS box can be checked and the value entered. If the RITIS FFS is less than 45 mph or if the posted speed limit is 40 mph or less then the FFS will need to be calculated from segment data (e.g. lane/shoulder width, heavy vehicles, access points) and any RITIS/field-measured value discarded.

Most segment data can be entered into HCS directly except for access points. Access points are calculated by including approaches in both directions unless a median barrier

or median striping forces traffic to one side of the roadway. So, if a barrier prevents eastbound traffic from entering a driveway on the north side of the street, that driveway is not counted in the total number of access points for the eastbound analysis direction. It would be counted for the westbound analysis direction, however.

Overall, commercial and industrial driveways and residential driveways that serve three or more dwellings should be counted in the total. Skip single family/duplex driveways, dirt roads/rural minor collectors, or field/emergency accesses where daily traffic is estimated to be 20 vpd or less.

Step 5: Estimation of the average (travel) speed

The majority of Step 5 involves calculating the effects of horizontal curvature and vertical terrain class. As mentioned previously, the ODOT [Horizontal Grade Report](#) (for state highways) will be necessary to identify horizontal curves that are needed to properly segment the facility.

Exhibit 11-20 shows a typical (exported to Excel version) Horizontal Grade Report output. The file extension will need to be renamed in Excel to import it properly. The report is laid out in a somewhat awkward two-line group, so the header and each record will require some cleanup. The exhibit shows the report with some of the excess blank rows removed, some spiral columns removed, and cell borders and shading added.

Exhibit 11-20 Exported Horizontal Grade Report (Formatted)

Roadway	Mileage Type	Overlap Mileage	Beginning Mile Point	Contract ID	Location Seq #	Horizontal Bearing	Est Data	Super Elev Rate	Spirals	
	Degree Curve Ang	Dir Code	Central Ang	Curve Len	Tangent Len	Total Ang	Curve Type		Len	Ang
1			85.11	16	140	0	C	0	0	0
	0		0	0	0	0	T		0	0
1			84.83	16	140	0	C	0	500	8.5
	3.4	L	27.35	804.41	402.68	39.25	S		200	3.4
1			84.61	16	140	0	C	0	0	0
	0		0	0	0	0	T		0	0
1			84.43	16	140	0	C	0	400	4
	2	R	3.0278	151.39	75.73	11.0278	S		400	4

The top header row corresponds with the first line of information and includes the typical highway segment identification data (e.g. roadway designation, beginning milepoint, and contract identification/highway number; Santiam Hwy #016 in this case), the superelevation rate, and the spiral “in” transition length. The second row contains the necessary curve geometry data (e.g. central angle in degrees, curve length in feet) which

is needed to calculate the curve radius needed by HCM Exhibit 15-22 to obtain the horizontal curve classification along with the spiral “out” transition length.

To calculate the curve radius from the data given in the Horizontal Grade Report, use the following formula:

$$R \text{ (ft)} = (180 \times L_c) \div (\pi \times \Delta)$$

Where:

R = curve radius (ft)

L_c = Length of curve (ft)

Δ = Central angle (degrees)

Curves with radii greater than 2550' should be ignored as these are too large to affect the speed of vehicles in the HCM procedure as shown in HCM Exhibit 15-22 and are considered tangent sections. Example 11-14 shows an example of these calculations.

When entering subsegments into HCS segments, the overall segment length will need be parsed across all curve or tangent portions. Use the beginning milepoints of the curves and adjacent tangent sections from the Horizontal Grade Report to figure out the curve subsegments. This way, the length of the circular curve and the spiral transitions are automatically included. Alternately, the spiral in & out lengths can be summed together and added to the circular curve length to obtain a total curve segment length. These should be listed in the analysis direction in order that they occur (e.g. tangent, curve, tangent).

Example 11-14 Calculating Horizontal Curve Radius & Classification

This example uses data from Exhibit 11-20 which was imported data from the Horizontal Grade Report for a section of US20, Santiam Highway #016. Information from that table is needed to calculate the corresponding curve and spiral radii.

Using Exhibit 11-20 for MP 27.53, the curve length is 804.41 ft, and the central angle is 27.35 degrees.

Using the formula:

$$R \text{ (ft)} = (180 \times L_c) \div (\pi \times \Delta)$$

$$\text{Radius} = (180 \times 804.41) / (3.14159 \times 27.35) = 144793.8 / 85.92 = 1685.2 \text{ ft}$$

In this case, the resulting curve radius is less than 2550 ft, so curve and tangent subsegments will be necessary within the larger segment that this curve falls into. In Exhibit 11-20, the leading (spiral in) transition is 500 feet, and the trailing (spiral out) transition is 200 feet which needs to be added to the total curve length (i.e. $500 + 200 + 804.41 = 1504.41$ feet). Note that this distance is also 0.28 mi which is the same as the

distance between the starting milepoint of the curve (MP 27.53) and adjacent downstream tangent section (MP 85.11)

In addition, once the curve radius is obtained, the superelevation rate is used in HCM Exhibit 15-22 to determine the horizontal curve classification. The superelevation rate in this case is 0.00 (0%). A curve with 1504.4 feet and 0% superelevation is entered into HCS for that curve subsegment that is used to determine the average speed for each curve segment in following steps.

For the curve at MP 84.43, the curve length is 151.39 ft and the central angle is 3.0278 degrees as shown in Exhibit 11-20.

Using the formula:

$$R (ft) = (180 \times L_c) \div (\pi \times \Delta)$$

$$\text{Radius} = (180 \times 151.39) / (3.14159 \times 3.0278) = 27250.2 / 10.3427 = 2634.7 \text{ ft}$$

In this case, the curve radius is greater than 2550' so this curve should be ignored and the length assumed to be tangent.

Step 6: Estimate Percent Followers

For each segment, the total percent followers are estimated along with the percent followers at capacity and of 25% of capacity. Most of the effort in this step is computing the various coefficient values using HCM Equations 15-17, 18, & 20 along with HCM Exhibits 15-24 & 26. This is done automatically in HCS.

Step 7: Calculate Additional Performance Measures Values for a Passing Lane Segment

Step 7 is skipped as this is only for passing lane segments which should not be in a dense Class III segment.

Step 8: Estimate Follower Density

HCS will use HCM Equation 15-35 to determine the follower density of the Class III segments. This uses the analysis directional flow rate, the average segment speed including any horizontal curve effects from Step 5 and the total percent followers from Step 6.

Step 9: Determine Potential Adjustment to Follower Density

Step 9 is only necessary if there is an adjacent (upstream) passing lane to the first Class III segment. It is common to have passing lanes immediately outside communities, so the effective length of the passing lane may have an influence on the downstream Class III segment (although the density of land uses and related access points may mute the effect somewhat). If there is an adjacent passing lane to the Class III segment, make sure that the segment analysis is expanded to cover this section. HCM Equations 15-36 through 38 are used to determine the effective length of the upstream passing lane and its downstream effect.

Step 10: Determine LOS

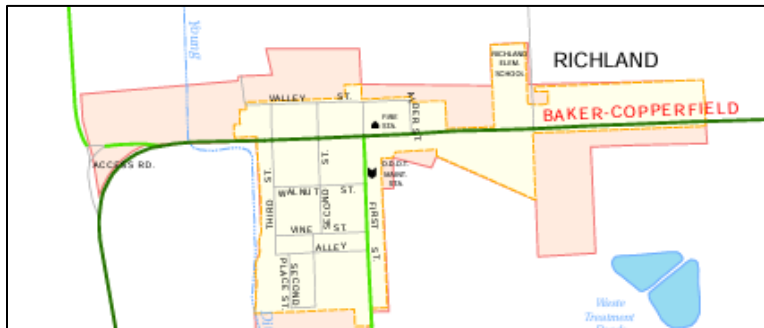
Use Exhibit 11-19 to compare the follower density from Step 8 or 9 to the thresholds to obtain a segment LOS.

Step 11: Calculate Facility Follower Density & LOS

Step 11 only applies if there are multiple segments making up the Class III section. As mentioned in Step 1, these could be based on various input parameter changes such as speeds and lane width or the presence of vertical grades and horizontal curves. Example 11-15 provides an example for assessing LOS on Class III highways section.

Example 11-14 Class III Highway LOS

This example demonstrates the application of follower density-based LOS criteria for a Class III highway to find the expected LOS in the eastbound direction on the two-lane highway segment through Richland on OR86 Baker-Copperfield Highway #012 as described below:



Input Data

- Baker – Copperfield Highway (OR86: Hwy #012), MP 41.06- 42.65
- Peak hour volume = 115 veh/h (total in both directions; 2025 data)
- Directional split (during analysis period) = 58% EB and 42% WB

- PHF = 0.85
- 27 % trucks in both directions
- Level terrain, no grades
- Field measured FFS = 31 mph

Geometric Segment Data¹

Seg #	BMP	EMP	Shoulder Width (ft)	Speed Limit (mph)	Passing	# Access Points
1	41.06	41.09	0	40	Constrained EB	1
2	41.09	41.28	0	40	Constrained Both Dir	1
3	41.28	41.33	0	25	Constrained Both Dir	0
4	41.33	41.38 ²	12	25	Constrained Both Dir	0
5	42.00 ²	42.22	8	25	Constrained Both Dir	8
6	42.22	42.27	3	25	Constrained Both Dir	0
7	42.27	42.40	3	40	Constrained Both Dir	2
8	42.40	42.50	1	40	Constrained WB	2
9	42.50	42.65	1	55	Passing Zone	0

¹From Digital Video Log, ODOT Highway Inventory Reports

²Milepoint equation at this location (MP 41.38 = MP 42.00)

LOS Methodology

This highway segment is a rural major collector and travels through the town of Richland. This will have local traffic mixing with the through traffic, has a higher amount of access points, no signalized intersections, a curbed cross-section, and has reduced speed limits. Since there are no signalized intersections or a two-way left turn lane median, the two-lane methodology still applies and falls into the Class III categorization which will require the HCM methodology.

Facility segmentation

This approximately 1.6-mile section of OR86 covering the UGB limits of Richland needs to be broken into nine segments based on changes of shoulder width, speed limit and passing opportunities as shown in the table above.

There is one horizontal curve starting at MP 41.01 to 41.26 which would affect Segments 1 & 2. From the ODOT Horizontal Grade Report this curve is 57.0417 degrees with a simple (circular) curve length of 570.42 ft. The curve is also bracketed by 400 ft spiral transition curves which make the total curve length 1370.42 ft which is accounted for already by the beginning milepoints of the curve (i.e. MP 41.01) and the adjacent tangent section (i.e. MP 41.26). The circular curve length is used to help calculate the curve radius, while the total curve length is used to calculate the curve subsegment lengths.

Using the formula:

$$R \text{ (ft)} = (180 \times L_c) \div (\pi \times \Delta)$$

$$\text{Radius} = (180 \times 570.42) / (3.1415 \times 57.0417) = 573 \text{ ft}$$

Determine demand flow rates and d/c ratio check

The directional demand flow rate in the analysis direction and the opposing direction with the matching peak hour factors (PHF) were calculated using HCM Equation 15-1.

$$v_i = \frac{V_i}{PHF}$$

For each analysis direction, the volume is the total two-way volume multiplied by the directional factor. These volumes are the ones entered into HCS.

$$\text{EB directional volume} = (115 \text{ vphpl} \times 0.58) = 67 \text{ vphpl}$$

$$\text{WB opposing volume} = (115 \text{ vphpl} \times 0.42) = 48 \text{ vphpl}$$

The d/c ratio check is performed by dividing each directional volume by the PHF to convert to a flow rate then divide it by the capacity of the section (1700 vphpl).

$$\text{EB d/c} = (67 \text{ vphpl} / 0.85) / 1700 \text{ vphpl} = 0.05 < 1.01 \text{ OK; not LOS F}$$

$$\text{WB d/c} = (48 \text{ vphpl} / 0.85) / 1700 \text{ vphpl} = 0.03 < 1.01 \text{ OK; not LOS F}$$

Since the d/c for both directions is not greater than 1.00, then the analysis can continue by entering the segment data into HCS for the eastbound direction.

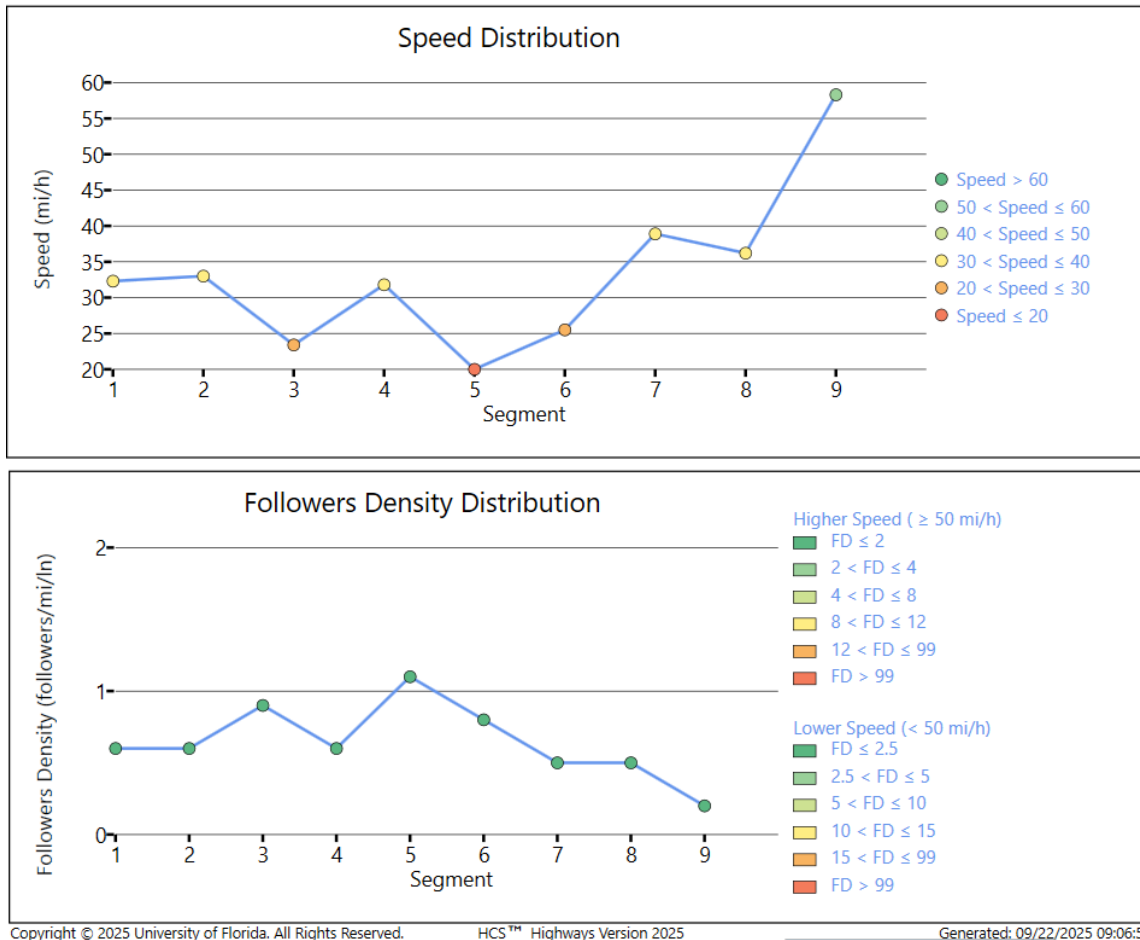
HCS two-lane facility analysis

The global inputs were entered that would not change between segments. This included PHF, directional and opposing demand volumes, lane width, heavy vehicles and grade. Then the nine segments were created, and the passing type, length, and speed limit were

modified according to the data table above. The details for shoulder width and access point density (divide the number of accesses by the segment length) were added to all segments. Lastly, curve subsegments with the noted curve radius and length were added to Segments 1 & 2. A subsegment was added to Segment 2 as the curve only goes to MP 41.26 but the segment extends to MP 41.28.

Facility Analysis Results

From the HCS figures below, horizontal curves (Segments 1-2), narrower shoulders (Segments 1-3 & 6-9) and higher access point density (Segment 5) causes operating speeds to decrease while the opposite will increase speeds. Lower speeds will also cause the follower density to increase.



The analysis also shows that the follower density is equivalent to a LOS A for the section.

Facility Results					
VMT veh-mi/AP	VHD veh-h/AP	Follower Density followers/mi/ln	Travel Time min	Space Mean Speed mi/h	LOS
16	0.00	0.6	1.90	30.3	A

The analysis would be repeated for the WB direction to complete the analysis for the section.

11.4.5 Passing and Climbing Lanes

Both passing and climbing lanes are low-cost improvements that can be very effective in improving the operation of two-lane highways and can reduce the need to widen highways to four lanes. The *HCM* includes methodologies for analyzing these types of facilities in Chapter 15. APM Appendix 10A includes criteria for helping to determine if these sections operate as auxiliary lanes (no increase in systemic capacity) or as multilane highways.

When analyzing either future build passing or climbing lanes it must be determined whether a no-passing restriction will be placed on opposing traffic in the area of the added lane. If passing by opposing traffic will not be allowed or is unknown at the time of analysis, the operations of opposing traffic must be analyzed for both cases.

The facility-level analysis described below can help identify areas of where passing or climbing lanes may be beneficial and Appendix 10A can help to indicate a potential starting spot for length. Actual conditions, including maximum grades, intersecting roadways, and access points may impact the placement. Refer to [HDM](#) Section 221 on information for maximum grades, Section 328 on climbing lanes, and Section 329 on passing lanes. Work with Region Traffic to identify appropriate locations and lengths to use for design.

Passing Lanes

Passing lanes are typically used where there may be inadequate passing opportunities, either because of sight distance limitations or as traffic volumes approach capacity. By providing a safe place to pass, passing lanes tend to reduce unsafe passing maneuvers. In addition to improving operations in the segment containing the passing lane, operations of the highway downstream of the passing lane may also be improved for some distance, known as the effective length, but not any longer than the start of the next passing (or climbing lane). The *HCM* methodology is applicable to directional segments of two-lane highways that include the entire passing lane and should also include the full effective downstream length, if possible. Refer to HCM Chapter 15 for more information. Example 11-16 shows an application with passing lanes of the methodology using HCS two-lane highway facility analysis.

When using the Highway Capacity Software (HCS) to perform calculations for a passing lane, a facility-level analysis should be employed as the section will need to be broken into multiple segments. Unlike the Class III analysis in the previous section, passing lanes are limited to rural sections with ideally no intersections or accesses (but this is not always the case). Segments will usually only break on shoulder width or passing type with subsegments indicated for horizontal curves. If a measured FFS is obtained from RITIS, then this will simplify the segmentation as shoulder width differences and access points are not needed.

Segments should be broken at intersections only if volumes change more than 10% otherwise, they can be taken as an access point instead. Since these analyses will be in rural locations, driveways or local roadways can be ignored unless it is shown that there would be more than 20 vpd (3+ dwellings).

At a minimum there will need to be at least three segments:

- Upstream of the passing lane.
- The passing lane, including tapers.
- Downstream of the passing lane covering as much as possible of the effective length, but no longer than the start of the next passing/climbing lane



The tapers, especially the downstream taper, deserves additional scrutiny and analysis to evaluate how traffic will merge back together given that it is common to have large speed differentials in these areas. Additionally, the downstream taper from a design perspective is almost more important than the passing lane itself since, unlike multilane facilities, there is no other lanes for traffic in the through lane to move to and create a gap. This will likely require use of alternative tools, such as microsimulation with Vissim, in order to have enough detail to be able to analyze these locations.

If volumes decrease by more than 10% between the passing lane and downstream segments, it may be necessary to also investigate testing the segments with a constant volume as the volume change may obscure the full effective length of the passing lane. The effective length is the point where the percent followers return to the same level as upstream of the passing lane or where the follower density reaches 95% of the upstream level.

If there are many volume changes, either increasing or decreasing, supplementing this analysis with microsimulation (Vissim) may be helpful to better judge the section operations.

Example 11-15 Passing Lane LOS

A section of eastbound OR58, Willamette Highway #018, from MP 6.50 to 14.40, including the subject passing lane from MP 7.10 – 7.93 was analyzed to determine its effectiveness. The next eastbound passing lane taper starts at MP 14.40 so the entire effective analysis length can be captured, however, it is not known if the actual effective length extends that far.

There were five noted volumes through the section, however the volume differences were less than 10% between MP 9.53 and 13.22 so these were averaged together. The table below shows the total and directional volumes, the directional factor and PHF so the

capacity check could be performed before entering data into HCS. All eastbound volumes were shown to be less than 1700 vphpl, so the analysis could proceed. The section will be broken for volume for these four sections and for heavy vehicles as applicable.

MP Range	Total Vol (vphpl)	D-factor	EB Volume (vphpl)	PHF	d/c <1.0?	HV %
6.50 – 8.55	1613	0.51	865	0.95	Yes	5.5
8.55 – 9.53	1304	0.51	700	0.95	Yes	3.3
9.53 – 13.22	1035	0.51	556	0.95	Yes	3.3
13.22 -14.47	870	0.51	467	0.95	Yes	9.4

The posted speed through the section is 55 mph. The measured FFS was obtained from RITIS' Performance Charts module and was 57 mph for the eastbound direction. Since a measured FFS was obtained, lane width, shoulder width, or access point data will not need to be gathered.

Passing-constrained (no passing) in the eastbound direction, the passing lane (including tapers), and the any passing zones were also noted for the analysis section. Short passing zones were combined into adjacent passing constrained section to minimize sections under 1320'. Segment 5 is short because of the volume breaks so any segment length warnings there will be ignored.

Seg #	MP Range	Segment Length (ft)	Passing Type
1	6.50 – 6.78	1478	Passing Zone
2	6.78 – 7.10	1690	Passing Constrained
3	7.10 – 7.99	4699	Passing Lane
4	7.99 – 8.55	2957	Passing Constrained
5	8.55 – 8.74	1003	Passing Constrained
6	8.74 – 9.53	4171	Passing Zone
7	9.53 – 10.78	6600	Passing Zone
8	10.78 – 11.20	2218	Passing Constrained
9	11.20 – 12.22	5386	Passing Constrained
10	12.22 – 12.74	2746	Passing Zone
11	12.74 – 13.22	2535	Passing Constrained
12	13.22 – 13.73	2693	Passing Constrained
13	13.73 – 14.40	3537	Passing Constrained

The Vertical Grade Report was also checked for the highway section. No significant grades were noted other than for a 3% grade for the entirety of Segment 8 and a 5% grade starting at MP 13.73 to the end of the section. Segment 13 was created to accommodate the change in grade. All other sections were coded at 0% grade.

The Horizontal Grade Report was also reviewed for the section, and nine curves were noted for creating future subsegments. The affected segment, MP, central angle, and

curve length were captured in the table below. The superelevation rate was zero in all segments, so it was not included in the table. The curve radius was then calculated as per Example 11-15 and shown in the table below. Review of the calculated radii indicated that the curves in Segments 4, 6, and 9 exceed the 2550 ft limit, so these can be ignored and treated as tangent sections when creating the subsegments.

Seg #	MP	Central Angle (deg)	Curve Length (ft)	Curve Radius (ft)
4	8.23	1.4475	374.02	14805
6	9.02	9.7331	1946.66	11459
9	11.41	26.7667	1338.33	2865
9	11.94	16.2167	810.83	2865
11	12.78	19.7581	493.96	1432
12	13.56	17.1372	285.62	955
13	13.75	21.0917	351.53	955
13	14.05	8.7581	145.97	955
13	14.25	44.6831	558.54	716

For subsegments, the affected segment total length was parsed into adjacent curve and tangent lengths. The milepoints for the curve and adjacent tangent sections were used to develop the subsegments so the spiral transition lengths are included in the curve length. The segment number and length, curve type, subsegment length and curve radius are shown in the table below to make HCS data entry more efficient.

Segment	Segment Length (ft)	MP	Curve Type	Subsegment Length (ft)	Curve Radius (ft)
11	2535	12.74	Tangent	211	n/a
		12.78	Curve	1479	1432
		13.06	Tangent	845	n/a
12	2693	13.22	Tangent	1795	n/a
		13.56	Curve	845	955
		13.72	Tangent	53	n/a
13	3537	13.73	Tangent	106	n/a
		13.75	Curve	950	955
		13.93	Tangent	634	n/a
		14.05	Curve	739	955
		14.19	Tangent	316	n/a
		14.25	Curve	792	716

A number of the data elements were entered as global inputs into the HCS two-lane facility module to reduce the amount of data entry required. These included FFS, PHF, speed limit, and grade. Passing type, segment length, volumes, grade, and heavy vehicles were added as each segment was created. Lastly, the “Details” window was filled in for each segment which was limited to adding in the curve subsegments as a measured FFS was obtained.

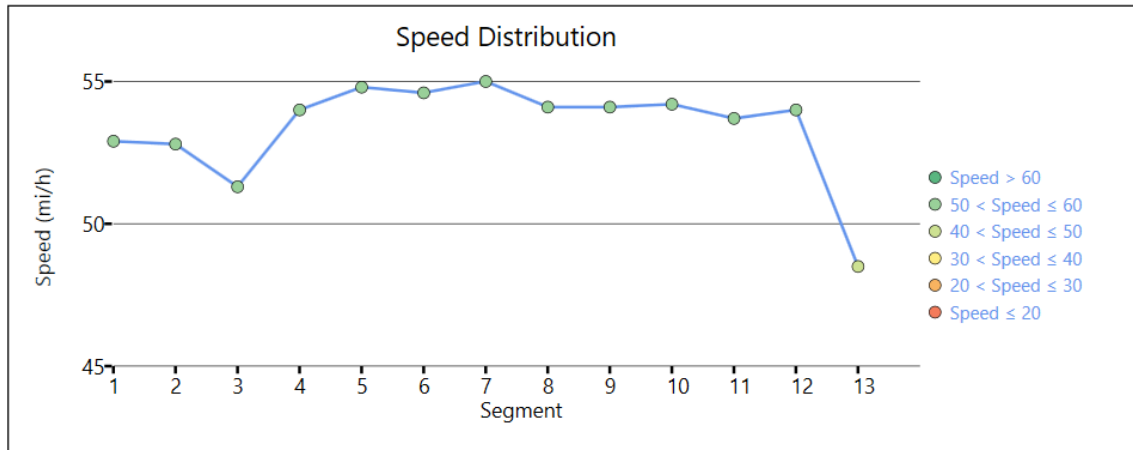
LOS results varied throughout the section from B to E. Upstream of the passing lane, the LOS was E in Segments 1 & 2, LOS B in the passing lane (Segment 3) and LOS D immediately downstream of the passing lane in Segment 4. From Segments 5 to 13 the volume drops, and the follower density decreases throughout even as there is increasing influence from grades and tighter curves. The table below shows the segment d/c ratio, speed, follower density and LOS from the HCS reports.

Seg #	D/C	Speed (mph)	Follower Density (followers/mi/ln)	LOS
1	0.54	52.9	12.5	E
2	0.54	52.8	12.6	E
3	0.61	51.3	12.0	B
4	0.54	54.0	9.9	D
5	0.43	54.8	7.4	C
6	0.43	54.6	7.3	C
7	0.34	55.0	5.2	C
8	0.34	54.1	5.9	C
9	0.34	54.1	5.7	C
10	0.34	54.2	5.6	C
11	0.34	53.7	5.9	C
12	0.29	54.0	4.5	C
13	0.29	48.5	6.3	C

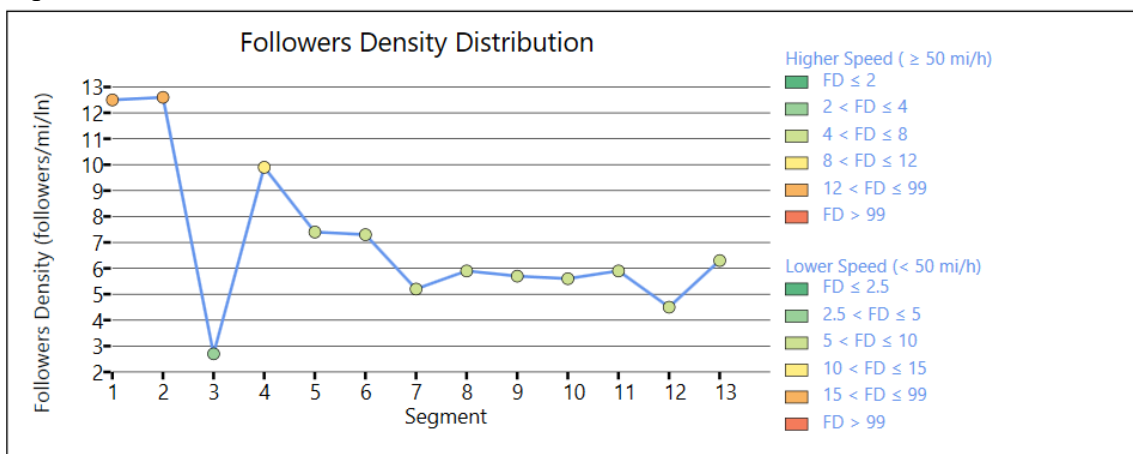
The overall facility section LOS was C as shown in the summary results:

Facility Results					
VMT veh-mi/AP	VHD veh-h/AP	Follower Density followers/mi/ln	Travel Time min	Space Mean Speed mi/h	LOS
1266	1.54	6.3	8.90	53.3	C

This graph shows the impact of the improvement in speed in Segments 4-12 downstream of the passing lane (however the volume also drops in Segment 5). The impact of the grade in Segment 13 is clearly shown which would likely be alleviated by the climbing lane immediately after that segment.



The last graph shows the substantial impact of the passing lane on follower density which is what the LOS is based on. Downstream of the passing lane, the follower density in Segment 4 is still less than in Segment 1 & 2, however, the volume drops obscure the impact.

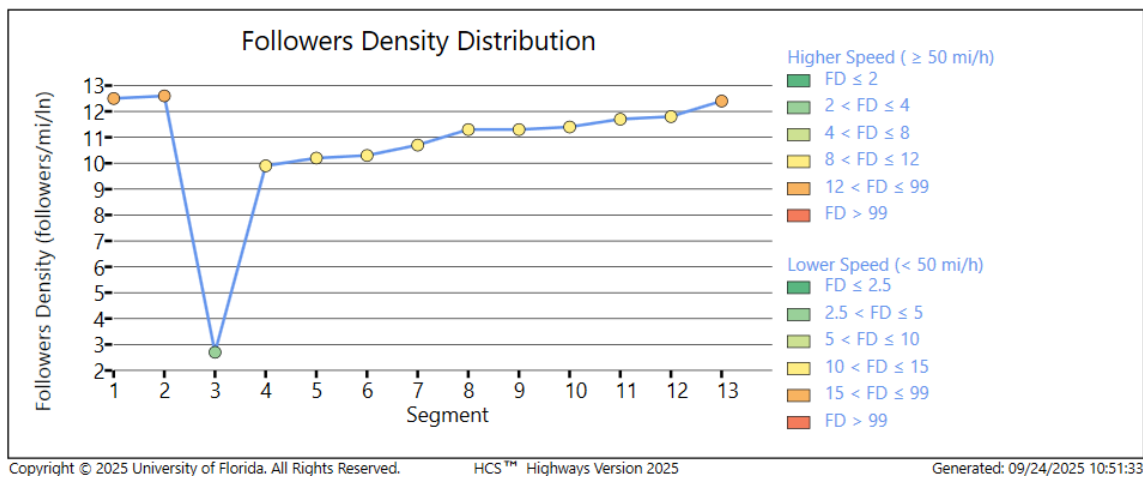


Copyright © 2025 University of Florida. All Rights Reserved. HCS™ Highways Version 2025 Generated: 09/24/2025 10:05:55

As a test, the highest section entry volumes (865 vph EB, 748 vph WB) were made global inputs to see the true impact of the passing lane's effective length downstream. The table below compares the upstream (Segment 2) percent followers and follower density with the segments downstream of the passing lane.

The effective length would normally be the shorter where percent followers are equal to the upstream or when the follower density is within 95% of the upstream value. This point seems to occur at the end of Segment 4 at MP 8.55, but this section is short compared to the whole section and does not seem to be consistent with following sections as the percentage improves instead of staying at zero or worse. In this case the effective length should be based on the follower density. The follower density does not exceed 95% of the upstream until Segment 13 as also shown in the graph below which would be about 5.7 miles.

Seg #	% Followers Improvement	Follower Density Comparison (%)
4	1.7	78
5	-1.0	81
6	3.4	82
7	4.1	85
8	0.8	90
9	3.0	90
10	2.5	90
11	1.2	93
12	1.5	94
13	2.2	98



Climbing Lanes

Climbing lanes are similar to passing lanes but are generally used where grades cause unreasonable reductions in operating speeds of some vehicles. An unreasonable reduction in operating speeds is typically considered to occur where speed differentials of more than 10 mph are created. These lanes increase the local capacity of a two-lane highway section (but not the whole highway) by providing a specific lane for slower vehicles to travel in while climbing an extended grade. This enables faster vehicles to pass these slower vehicles safely without having to leave the main travel lane. While climbing lanes are typically thought of as being associated with upgrades, they can also be applied to downgrades where heavy vehicles must drive in a low gear to avoid speeding out of control.

Analysis of climbing lanes are like passing lanes with the same sort of segmentation considerations. There should be at least one segment upstream of the climbing lane and at least one (likely more) downstream of the climbing lane. If it is desired to figure out the full effective length of the climbing lane, then multiple downstream segments will be necessary up to the next passing/climbing lane in a particular direction.

There may be multiple grade changes, so more segments may be necessary to capture the vertical profile. As with passing lanes, obtaining a measured FFS from RITIS will reduce the number of segments as shoulder and lane width changes and access points are not needed.

Example 11-16 Potential Climbing Lane Analysis

A section of westbound US20, (Santiam Highway, 016) from MP 84.16 to 86.77 was analyzed to determine if the existing climbing lane starting at MP 84.16 should be extended east to the bottom of the Santiam Pass upgrade approximately at MP 86.77 in a future 2045 project. The resulting speed of the typical design truck (WB-67 semi-trailer truck) would need to be compared with the operation speed of the section to determine if the speed differential would exceed 10 mph.

This section has 12' lanes, 6' shoulders, no access points, and 55 mph over the entire length. Volumes were estimated from available 2024 AADTs to be 1005 vph with 52% in the westbound direction. The Future Highway Volume Table indicated 12% growth between 2024 and 2045 which results in 1126 vph (586 vphpl WB, 540 vphpl EB) which are below the 1700 vphpl threshold. A count-based PHF nearby was 0.88 and the heavy truck percentage is 7.1%.

Passing-constrained (no passing) and any passing zones were also noted for the westbound (decreasing milepoints) analysis section. Short passing zones were combined into adjacent passing constrained section to minimize sections under 1320'.

Seg #	MP Range	Segment Length (ft)	Passing Type
1	86.77 – 86.44	1742	Passing Constrained
2	86.44 – 85.45	5227	Zone
3	85.45 – 84.90	2904	Passing Constrained
4	84.90 – 84.63	1426	Passing Zone
5	84.63 – 84.16	2481	Passing Constrained

The Vertical Grade Report was also checked for the highway section. There is a 2.1% upgrade from MP 86.77 to 86.52 and a 5% upgrade from MP 86.52 to 84.16. To avoid creating a short segment the change from 2.1 to 5% was placed at MP 86.44.

The Horizontal Grade Report was also reviewed for the section, and seven curves were noted for creating future subsegments. The affected segment, MP, central angle, and curve length were captured in the table below. The superelevation rate was zero in all segments, so it was not included in the table. The curve radius was then calculated as per Example 11-15 and shown in the table below. Review of the calculated radii indicated that the curves in Segments 1 through 3 and 5 exceed the 2550 ft limit, so these can be ignored and treated as tangent sections when creating the subsegments.

Seg #	MP	Central Angle (deg)	Curve Length (ft)	Curve Radius (ft)
1	86.77	5.2500	249.02	2718
1,2	86.54	9.9831	665.55	3820
2	86.35	8.3167	415.03	2859
2	85.68	3.4167	341.67	5730
3	85.14	13.0667	1187.88	5209
4	84.83	27.3500	804.41	1685
5	84.43	3.0278	151.34	2864

For subsegments, the affected segment total length was parsed into adjacent curve and tangent lengths. The milepoints for the curve and adjacent tangent sections were used to develop the subsegments so the spiral transition lengths are included in the curve length. The curve in Segment 4 overlaps into Segment 3, so a subsegment was required so the full effect of the curve could be accounted for. The segment number and length, milepoint, curve type, subsegment length and curve radius are shown in the table below to make HCS data entry more efficient.

Segment	Segment Length (ft)	Milepoint	Curve Type	Subsegment Length (ft)	Curve Radius (ft)
3	2904	85.14	Tangent	1795	n/a
		84.90	Curve	1109	1685
4	1426	84.83	Curve	370	1685
		84.83	Tangent	1056	n/a

Several of the data elements were entered as global inputs into the HCS two-lane facility module to reduce the amount of data entry required. These included FFS, PHF, speed limit, and grade. Passing type, segment length, volumes, grades, and heavy vehicles were added as each segment was created. Lastly, the “Details” window was filled in for each segment which was mainly adding in the appropriate curve subsegments as the lane, shoulder and access point values were the same.

LOS results remained constant throughout the facility section at C which appears to indicate acceptable operation of the current cross-section. The table below shows the segment d/c ratio, speed, follower density and LOS.

Seg #	D/C	Speed (mph)	Follower Density (followers/mi/ln)	LOS
1	0.39	58.9	7.4	C
2	0.39	54.7	7.9	C
3	0.39	56.4	7.5	C
4	0.39	59.0	7.2	C
5	0.39	56.6	7.5	C

Climbing lanes should be considered for a section if the expected truck speed reduction (i.e. speed differential) exceeds 10 mph. The Climbing Lanes portion of the HCS text

report shows the initial overall entering speed for all vehicles and the speed reductions for different truck classes by segment. The “Interstate TT” (tractor-trailer) will be the most typical truck and is closest to assumed design vehicle of an improvement.

Seg #	Initial Speed (mph)	Interstate Truck Speed Reduction (mph)	Consider Climbing Lane? (>10 mph differential)
1	58.9	0.0	No
2	58.9	18.3	Yes
3	54.7	15.0	Yes
4	56.4	10.1	Yes
5	59.2	17.1	Yes

From the truck speed results, an extension of the climbing lane could be considered as a future project covering the 5% grade portions of the overall analyzed facility. This would be a 2.28-mile extension covering Segments 2 – 5 from MP 86.44 to MP 84.16.

11.5 Travel Time Reliability

As described in APM Section 9.3, travel time reliability (or simply, *reliability*) considers (1) the range of potential travel times roadway users may experience, (2) the consistency of travel times, and (3) the ability of a roadway to provide a desired travel time. APM Section 9.3.5 described resources available for evaluating and reporting reliability under existing conditions. This section presents methods for forecasting future travel time reliability.

11.5.1 Overview of Travel Time Reliability Methods

APM Section 9.3.6 introduced the primary types of methods for forecasting reliability:

- Planning-level methods based on the SHRP 2 C11 equations for estimating common reliability performance measures;
- Detailed macroscopic methods, which develop a travel time distribution from the results of hundreds of scenarios modeling different demand levels, weather conditions, incidents, work zones, and special events; and
- Microscopic simulation.

This subsection summarizes the methods presented in Section 11.5 and describes potential applications for them.

Planning-Level Method

PPEAG-Based Screening Method

The screening method described in Section 11.5.3 also uses the SHRP 2 C11 equations to estimate common reliability performance measures but uses PPEAG-based methods to develop the v/c ratios and average speeds required as inputs to the equations. The method is well-suited to preliminary evaluations of the reliability of individual facilities, as well as for testing the reliability effect of projects that change roadway demand or capacity. It can be applied to any roadway type. Because screening methods are not sensitive to the effects of most roadway operations strategies, nor are they sensitive to weather conditions or facility-specific crash or incident rates, the detailed macroscopic methods described below should be used for detailed evaluations and alternatives analysis.

Detailed Macroscopic Methods

HCM Reliability Methods

HCM methods also define hundreds of scenarios and combine the results into a travel time distribution. The HCM methods offer more flexibility for defining a reliability reporting period and for testing roadway operations strategies, and they produce travel time estimates based on state-of-the-art HCM 7th Edition methods. The HCM method requires considerable work to gather and enter data about demand volumes and roadway characteristics (if not already performed as part of a traditional roadway operations analysis), but has a reasonable process for defining scenarios and generating a travel time distribution. HCM methods can only be used at present to evaluate the reliability of freeways and urban streets.

Simulation

As discussed in Section 9.3.6, although the FHWA has sponsored case studies demonstrating the potential use of microsimulation to forecast reliability, it is not practical to do so at present because of the time required to develop, code, run, and analyze the many different scenarios required to generate an accurate travel time distribution. Although it is relatively easy to batch-run changes in demand through a simulation model, other factors that influence reliability, such as weather, incidents, work zones, and roadway operations strategies require coding and running individual models. The FHWA case studies involved 8 or 9 common scenarios, whereas the HCM recommends a minimum of 200 scenarios for generating a travel time distribution and generating reliability performance measures. In particular, it is the effect of relatively rare scenarios that has the greatest influence on the overall reliability results. Consequently, simulation is not recommended at present as a tool for forecasting reliability.

Reliability Method Comparison

Exhibit 11-19 summarizes the analysis capabilities, data needs, and potential applications for the primary methods available for forecasting reliability.

Exhibit 11-19 Reliability Forecasting Method Comparison

	Screening	HCM	Simulation
<i>Analysis Needs and Data Availability</i>			
Preferred roadway types	Any	Freeways, urban streets	Any
Reliability reporting period	Weekday any hours, 1 year	Any	Weekday peak period, 1 year
Reliability performance measures	Common	Any	Difficult
Maximum study area size	Facility	Facility	Facility
Input data needs	Low	High	Very High
Defaults available for inputs	No	Yes	No
<i>Applications</i>			
Regional Transportation Plan	●		
Transportation System Plan	●		
Corridor Plan	●	●	
Refinement Plan	●	●	●
Project Development	●	●	●

Note: ○ = problem identification, ● = evaluation, ● = detailed evaluation.

11.5.2 Data Needs and Sources

Exhibit 11-20 summarizes the input data requirements for the primary methods available for forecasting reliability. In some cases, the methods calculate or obtain certain values directly without requiring the analyst to provide them. See Section 11.3 for suggested data sources for roadway section data, Appendix 11C for default values for reliability-related inputs, and Appendix 11F for guidance on developing local values for reliability inputs.

Exhibit 11-20 Input Data Required for Reliability Forecasting Methods

Input Data	Screening	HCM (Freeway)	Simulation
<i>Roadway Section Data</i>			
Free-flow speed	●	●	●
Posted speed	●	●	●
Volume-to-capacity ratio	●	*	
Number of lanes	●	●	●
Average speed	●	*	*
Section length	○	●	●
Detailed demand data		●	●
Detailed roadway geometry		●	●
Simulation parameters			●
<i>Reliability Factor Data</i>			
Demand levels	X	○	●
Incident types & durations	X	○	●
Severe weather types & durations	X	○	X
Adjustments for special events	X	○	●

Notes: ● = required input, ○ = optional input (can be defaulted), * = calculated value,
X = reliability factor not directly considered by method, empty = not required.

11.5.3 Screening Analysis Method

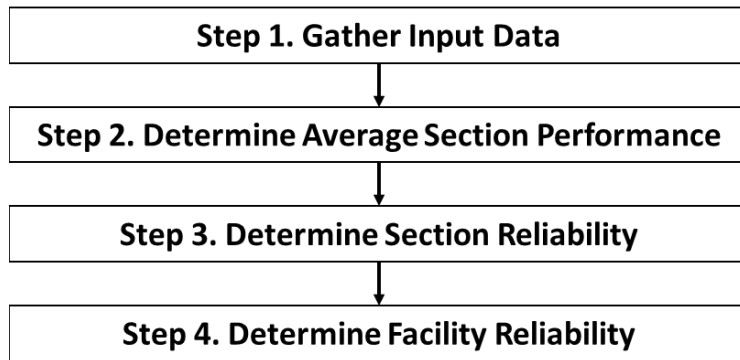
Introduction

The screening method can be applied to any roadway type. It is based on the “data poor” reliability forecasting equations developed by SHRP 2 Project C11, along with Oregon-specific modeling.⁵ Input values used by these equations are developed using PPEAG methods, which gives the screening method a greater sensitivity to local roadway conditions than does the basic SHRP 2 C11 method. The screening method first forecasts the reliability of individual roadway sections. The section results are then aggregated into a forecast of the reliability of a longer roadway facility.

⁵ Cambridge Systematics, Inc. Oregon SHRP2 C11 Reliability Analysis Implementation Plan, Task 2, Final Technical Memorandum #1. Medford, MA, Oct. 2018.

Calculation Process

The following steps are involved in estimating travel time reliability:



The method begins by determining average roadway performance on a section-by-section basis, following the screening methods described previously in APM Sections 11.3.1 through 11.3.4. Once each section's volume-to-capacity ratio and average peak-hour speed are determined, the method can then estimate any of the following performance measures for the section:

- Average (mean) travel time index (TTI_m , TTI_{Pm})
- 50th percentile TTI (TTI_{50} , TTI_{P50})
- 80th percentile TTI (TTI_{80} , TTI_{P80})
- 95th percentile TTI (TTI_{95} , TTI_{P95})
- Percent trips occurring at less than 45 mph
- Percent trips occurring at less than 30 mph
- Person hours of delay based on the posted speed

Step 1: Gather Input Data. The input data requirements are the same for performing a facility analysis. For freeway facilities, the following data are required for each section (items in italics can be defaulted):

- Section type (basic, merge–diverge, weaving)
- Hourly demand
- *Peak hour factor*
- *Percent heavy vehicles*
- Number of lanes
- *Free-flow speed*
- Posted speed
- Terrain class (level, rolling, mountainous)
- Section length

Step 2: Determine Average Section Performance. In this step, two key performance measures are determined for each section: volume-to-capacity ratio and average travel time without incidents. For freeway sections, the screening methods described in APM Sections 11.3.1 through 11.3.3 for basic freeway sections, merge–diverge segments, and weaving sections, respectively, are used to determine volume-to-capacity ratio. For other roadway types, use the corresponding PPEAG method to determine the volume-to-capacity ratio.

For freeway facilities, average travel time without incidents is estimated using a process similar to Step 7 of the screening method for freeway facilities (APM Section 11.3.4). However, because the reliability performance measure equations work with hourly volumes instead of 15-minute volumes, the Step 7 process is simplified to work with hourly volumes as follows:

$$T_{FFS,i} = \frac{3,600L_i}{FFS_i}$$

$$T_i = T_{FFS,i} + L_i(\Delta_{RU_i} + \Delta_{RO_i})$$

with

$$\Delta_{RU_i} = \begin{cases} 0 & X_i < E \\ A(X_i)^3 + B(X_i)^2 + C(X_i) + D & E \leq X_i \end{cases}$$

$$X_i = \min(1, d_i/c_i)$$

$$\Delta_{RO_i} = \frac{900}{2L_i} (\max[1, d_i/c_i] - 1)$$

where

$T_{FFS,i}$ = travel time to traverse section i at the free-flow speed (s),

L_i = length of section i (mi);

FFS_i = free-flow speed of section i (mph),

T_i = average travel time to traverse section i without incidents (s),

Δ_{RU_i} = undersaturated delay rate for section i (s/mi),

Δ_{RO_i} = oversaturated delay rate for section i (s/mi),

A, B, C, D, E = parameters from Exhibit 11-12 (see Section 11.3.4),

d_i = demand flow rate in section i (veh/h) = hourly demand divided by the peak hour factor, and

c_i = capacity of section i (veh/h).

For all other roadway types, use the corresponding PPEAG method to determine the average travel time without incidents.

Step 3: Determine Section Reliability. This step begins by estimating each roadway's mean travel time index as a function of the free-flow speed, the recurring delay rate, and the incident delay rate, as follows:

$$TTI_m = 1 + FFS \times (RDR + IDR)$$
$$RDR = \frac{1}{S} - \frac{1}{FFS}$$
$$IDR = [0.020 - (N - 2) \times 0.003] \times X^{12} \quad X \leq 1.00$$

where

TTI_m = mean travel time index (unitless)

FFS = free-flow speed (mph),

RDR = recurring delay rate (h/mi),

IDR = incident delay rate (h/mi),

S = average peak-hour speed (mph),

N = number of lanes ($N = 2, 3$, or 4 ; if there are more than 4 lanes, set N to 4), and

X = volume-to-capacity ratio (if $X > 1$, set X to 1).



Note that the estimate of incident delay rate used by the screening method is based on national averages and not individual roadway characteristics. If there is interest in evaluating the effect of safety countermeasures on roadway reliability, the detailed method should be used instead.

Once TTI_m is determined, additional reliability performance measures can be calculated from it. The choice of equations depends on the type of roadway being studied, and more performance measures can be estimated for some roadway types than for others. Four roadway categories are defined:

- Urban freeways and multilane highways,
- Rural freeways and multilane highways,
- Rural two-lane highways, and
- Urban arterials.

“Urban” and “rural” are determined using the FHWA definitions for federal-aid highways.

Urban freeways and multilane highways:

$$TTI_{50} = \frac{0.8701 \times 80.9980 + 14.0785 \times TTI_m^{2.2141}}{80.9980 + TTI_m^{2.2141}}$$

$$TTI_{80} = \frac{14.8892 \times TTI_m^{1.6443}}{5.0817^{1.6443} + TTI_m^{1.6433}}$$

$$TTI_{95} = 16.7754 \times \exp\left(-\frac{2.8221}{TTI_m}\right)$$

$$CONG_{30} = -9.1128 + 140.3250 \times \ln(TTI_m) \quad \text{if } TTI_m \geq 1.07; 0 \text{ otherwise}$$

$$CONG_{45} = -3.0184 + 205.3288 \times \ln(TTI_m) \quad \text{if } TTI_m \geq 1.02; 0 \text{ otherwise}$$

Rural freeways and multilane highways:

$$TTI_{50} = 1 + 0.5383 \times \ln(TTI_m)$$

$$TTI_{80} = 0.2834 \times e^{(1.2631 \times TTI_m)}$$

$$TTI_{95} = \frac{(0.9941 \times 21.0911 + 2.2971 \times TTI_m^{17.5709})}{(21.0911 + TTI_m^{17.5709})} \quad \text{with a minimum value of 1}$$

Rural two-lane highways:

$$TTI_{50} = 0.6836 \times \exp(0.3996 \times TTI_m)$$

$$TTI_{80} = \frac{30.0787}{1 + 75.7094 \times \exp(-0.9778 \times TTI_m)}$$

$$TTI_{95} = \frac{0.3691 \times 8.3171 \times 6.0980 \times TTI_m^{4.2633}}{8.3171 + TTI_m^{4.2633}}$$

Urban arterials:

$$TTI_{50} = \frac{0.5580 + 0.2236 \times TTI_m}{1 - 0.2618 \times TTI_m + 0.0307 \times TTI_m^2}$$

$$TTI_{80} = 0.5161 \times (TTI_m + 0.5105)^{1.6694}$$

$$TTI_{95} = \frac{9.1585}{1 + \left(\frac{TTI_m}{2.1327}\right)^{-2.8021}}$$

$$CONG_{20} = 7424.8705 \times \exp\left(\frac{-9.4124}{TTI_m}\right) \quad \text{for } TTI_m \geq 1.06; 0 \text{ otherwise}$$

where

TTI_{50} = 50th-percentile travel time index (unitless),

TTI_m = mean travel time index (unitless),

TTI_{80} = 80th-percentile travel time index (unitless),

TTI_{95} = 95th-percentile travel time index (unitless),

$CONG_{40}$ = duration of congestion, 40-mph threshold (minutes),

$CONG_{30}$ = duration of congestion, 30-mph threshold (minutes), and

$CONG_{20}$ = duration of congestion, 20-mph threshold (minutes).

ODOT uses a policy TTI based on the posted speed rather than the free-flow speed. The mean TTI is converted into a mean policy TTI as follows:

$$TTI_{pm} = TTI_m \times \frac{PSL}{FFS} \quad \text{with a minimum value of 1.00}$$

where

TTI_{pm} = mean policy travel time index (unitless),

PSL = posted speed limit (mph), and

all other variables are as previously defined. Similarly, TTI_{p50} , TTI_{p80} , and TTI_{p95} are found by multiplying TTI_{50} , TTI_{80} , and TTI_{95} , respectively, by the ratio of the posted speed to the free-flow speed.

Person hours of delay can also be determined for each roadway section as described in APM Section 9.3. The analyst first calculates the time to travel through the section at the posted speed. The portion of the estimated travel time that exceeds the travel time at the posted speed is then counted as delay. The number of persons experiencing the delay is then determined from vehicle volumes by type (e.g., passenger car, bus, truck) and an assumed occupancy for each vehicle type.

Step 4: Determine Facility Reliability. In this step, the mean travel time including incidents determined in Step 2 for each section is summed for the entire facility. Next, the mean TTI is determined for the facility and used to determine other reliability measures of interest for the facility, such as the 95th-percentile TTI for the facility. Finally, all TTI values are converted into the policy TTI values for facility.

The mean travel time to traverse the facility (including incidents) is determined as follows:

$$T_f = \sum_{i=1}^n T_{FFS,i} \times TTI_i$$

where

T_f = mean travel time to traverse the facility (s),

$T_{FFS,i}$ = time to traverse section i at the free-flow speed (s),

TTI_i = mean travel time index for section i (unitless), and

n = number of sections in the facility.

The mean facility travel time index $TTI_{m,f}$ is then:

$$TTI_{m,f} = \frac{T_f}{\sum_{i=1}^n T_{FFS,i}}$$

where all other variables have been defined previously.

The value of $TTI_{m,f}$ is then substituted for TTI_m in the equations in Step 3 to estimate other reliability performance measures of interest for the facility. Finally, all TTI results are multiplied by the ratio of the posted speed to the free-flow speed to develop the equivalent policy TTIs.

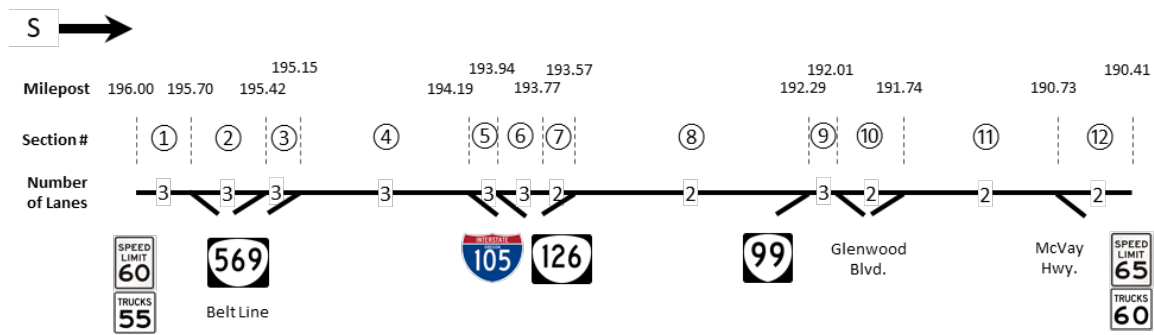
Example 11-17 Freeway Reliability Analysis (Screening Method)

The study facility is I-5 southbound through the Eugene urban area. The existing travel time reliability of I-5 is being assessed to provide a baseline condition for a transportation system plan that will eventually also include future-conditions assessments.

Step 1. Gather input data. The facility boundaries are set at the points where the automobile speed limit drops from 65 mph to 60 mph at the north end of the urban area, and where the speed limit changes back to 65 mph at the south end. On- and off-ramps are located at the milepoints shown in the diagram below; none of the ramps are metered.



©2019 Google



The freeway is divided into sections, with section boundaries located at ramp junctions. In addition, a section boundary is established at the point where the number of basic freeway lanes drops from 3 to 2. The section between the on-ramp from Highway 99 and the off-ramp to Glenwood Blvd. has an auxiliary lane and is therefore assigned as a weaving section. All other sections starting with an on-ramp are assigned as merge–diverge sections. All remaining sections are assigned as basic sections.

ODOT's *Traffic Volume Tables* are used to find directional AADTs for the I-5 mainline and ramp AADTs in the weaving section. ODOT's TransGIS tool (<https://gis.odot.state.or.us/transgis/>) is used to obtain K-factors, heavy vehicle percentages, and number of lanes in each section. The length of each section is determined from the milepoints of the section boundaries. Appendix 11C is used to obtain the following default values:

- Peak hour factor: 0.94 (urban freeway)
- Terrain: level (Figure 3)
- Ramp-to-ramp AADT in the weaving section: $285 \text{ veh} = (2,470 / 33,210) \times 3,830$

Figure 1 in Appendix 11B is used to obtain a capacity adjustment factor for driver population CAF_{pop} for the facility. That table gives a value of 0.968 (mostly familiar drivers) for I-5 in the Willamette Valley. There are no CAVs; therefore, the default capacity adjustment factor for CAVs CAF_{CAV} of 1.00 will be used.

The method provided in Appendix 11A for differential auto and truck speed limits is used to estimate the free-flow speed. For example, in section 1, the auto speed limit is 60 mph, the truck speed limit is 55 mph, and the heavy vehicle percentage is 24.4%. In the absence of a measured value, the auto free-flow speed is estimated as the auto speed limit plus 5 mph, or 65 mph. The truck free-flow speed is then estimated as the auto free-flow speed (65 mph) minus the difference in the auto and truck speed limits (5 mph), or 60 mph. Finally, a weighted average free-flow speed is calculated as follows:

$$FFS = (1 - P_T)FFS_{auto} + (P_T)FFS_{truck}$$

$$FFS = (1 - 0.244)65 + (0.244)60 = 63.8 \text{ mph}$$

The following table summarizes the input data by section.

Section	1	2	3	4	5	6	7	8	9	10	11	12
Type	B	B	MD	MD	B	B	B	MD	W	B	MD	B
# of lanes	3	3	3	3	3	3	2	2	3	2	2	2
AADT	24,010	17,190	23,990	36,110	29,470	20,070	20,070	30,740	33,210	29,380	30,640	26,640
K-factor	10.1	9.8	9.8	9.8	9.8	9.6	9.6	9.6	9.5	9.5	9.5	9.8
PHF	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Heavy vehicle %	24.4	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9	17.9
Length (mi)	0.30	0.28	0.27	0.96	0.25	0.17	0.20	1.28	0.28	0.27	1.01	0.32
FFS (mph)	63.8	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1	64.1
Posted speed (mph)	60	60	60	60	60	60	60	60	60	60	60	60
Terrain	Level	Level	Level	Level	Level	Level	Level	Level	Level	Level	Level	Level

Note: B = basic, MD = merge–diverge, W = weave.

Step 2. Determine Average Section Performance. Each section's volume-to-capacity ratio and average speed are determined following the procedures given in APM Sections 11.3.1 through 11.3.3. The process is illustrated for the weaving section (section 9).

- *Step 2.1: Develop hourly volumes.* AADTs are converted to hourly volumes by applying the section's K-factor. For example, the weaving section has an AADT of 33,210 and a K-factor of 9.5. The section's hourly volume is then $33,210 \times (9.5 / 100) = 3,155$ veh/h. Similarly, the on-ramp volume is 235 veh/h, the off-ramp volume is 364 veh/h, and the ramp-to-ramp volume is 27 veh/h. Because the ramp-to-ramp volume is known, the weaving volumes can also be determined. The ramp-to-freeway volume is the on-ramp volume minus the ramp-to-ramp volume: $235 - 27 = 208$ veh/h. The freeway-to-ramp volume is the off-ramp volume minus the ramp-to-ramp volume: $364 - 27 = 337$ veh/h.
- *Step 2.2: Adjust volumes.* The hourly volumes are next converted into 15-minute demand flow rates by dividing by the peak hour factor (PHF). For the weaving section, the PHF is 0.94 and the 15-minute flow rate is $3,155 / 0.94 = 3,356$ veh/h. Similarly, the ramp-to-freeway flow rate is 221 veh/h and the freeway-to-ramp flow rate is 359 veh/h.
- *Step 2.3: Determine capacity adjustment factors.* The CAFs for population (0.968) and CAVs (1.00, as there are no CAVs) have already been determined. There are no ramp meters; therefore, $CAF_{meter} = 1.00$. From APM Section 11.3.2, $CAF_{ramp} = 0.95$ for merge-diverge sections and 1.00 elsewhere. From APM Section 11.3.3, $CAF_{weave} = 1.00$ for all sections except the weaving section. The value of CAF_{weave} for the weaving section is a function of the weaving flows and the section length. The volume ratio VR is the sum of the ramp-to-freeway and freeway-to-ramp flow rates divided by the total flow rate in the weaving section:

$$VR = \frac{(221 + 359)}{3,356} = 0.173$$

The value of CAF_{weave} for the weaving section can now be determined using the equation found in APM Section 11.3.3:

$$CAF_{weave} = 0.884 - 0.0752VR + 0.0000243L_{weave}$$

$$CAF_{weave} = 0.884 - 0.0752(0.173) + 0.0000243(0.28 \times 5280)$$

$$CAF_{weave} = 0.907$$

- *Step 2.4: Determine section capacity and v/c ratio.* Each section's capacity can now be determined from the capacity equation given in APM Section 11.3.1 (basic sections), 11.3.2 (merge-diverge sections), or 11.3.3 (weaving sections).

For example, the weaving section's capacity is determined as follows, using a truck equivalency factor E_T of 2 for level terrain:

$$c = \frac{(2,200 + 10 \times (\min(70, FFS) - 50))}{1 + (E_T - 1)(\%HV/100)} \times N \times CAF_{weave} \times CAF_{pop} \times CAF_{meter} \times CAF_{CAV}$$

$$c = \frac{(2,200 + 10 \times (\min(70, 64.1) - 50))}{1 + (2 - 1)(17.9/100)} \times 3 \times 0.907 \times 0.968 \times 1.00 \times 1.00$$

$$c = 5,230 \text{ veh/h}$$

The v/c ratio is then the weaving section's flow rate divided by the capacity:
 $3,356 / 5,230 = 0.64$.

- *Step 2.5: Determine average section speed.* This step starts by calculating the average section travel time without incidents, as described in APM Section 11.3.4. The travel time is a function of the section length, free-flow speed, oversaturated delay rate, and undersaturated delay rate. Continuing with the example of the weaving section, the oversaturated delay rate is 0 because the section's v/c ratio of 0.64 is less than or equal to 1. The calculation of the undersaturated delay rate requires looking up as many as five parameters from Exhibit 11-12. First, the value of the E parameter for a free-flow speed of 65 mph is compared to the section's v/c ratio. The v/c ratio is greater than the E value of 0.62; therefore, there is some undersaturated delay and the other four parameters must be looked up. The undersaturated delay rate for the weaving section is calculated as:

$$\Delta_{RU_{i,t}} = A(X_{i,t})^3 + B(X_{i,t})^2 + C(X_{i,t}) + D$$

$$\Delta_{RU_{i,t}} = 92.45(0.64)^3 - 127.33(0.64)^2 + 56.34(0.64) - 8.00$$

$$\Delta_{RU_{i,t}} = 0.14 \text{ s/mi}$$

The mean travel time through the weaving section, without incidents, is then:

$$T_{i,t} = \frac{3,600L_i}{FFS_i} + L_i (\Delta_{RU_{i,t}} + \Delta_{RO_{i,t}})$$

$$T_{i,t} = \frac{3,600(0.28)}{64.1} + (0.28)(0.14 + 0)$$

$$T_{i,t} = 15.8 \text{ s}$$

Finally, the mean speed through the weaving section, without incidents, is:

$$S_{i,t} = \frac{3,600L_i}{T_{i,t}} = \frac{3,600(0.28)}{15.8} = 63.8 \text{ mph}$$

The following table provides travel times and mean speeds for each section along the study facility.

Section	1	2	3	4	5	6	7	8	9	10	11	12
T (s)	16.9	15.7	15.2	54.5	14.0	9.5	11.2	78.4	15.8	15.7	61.4	18.3
S (mph)	63.8	64.1	64.1	63.4	64.1	64.1	64.1	58.8	63.9	61.8	59.2	62.9

Step 3. Determine Section Reliability. This step begins by determining each section's mean travel time index TTI_m , accounting for the effects of incidents on the average section speed. The mean TTI is a function of the free-flow speed, the recurring delay rate, and the incident delay rate.

The incident delay rate is a function of the mean speed and the free-flow speed. For section #8, it is calculated as follows:

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

$$IDR = [0.020 - (2 - 2) \times 0.003] \times 0.86^{12}$$

$$IDR = 0.00327 \text{ s/mi}$$

For section #8, the recurring delay rate is:

$$RDR = \frac{1}{S} - \frac{1}{FFS} = \frac{1}{56.1} - \frac{1}{64.1} = 0.00141 \text{ s/mi}$$

Finally, the mean TTI for section #8 is calculated as follows:

$$TTI_m = 1 + FFS \times (RDR + IDR)$$

$$TTI_m = 1 + 64.1 \times (0.00141 + 0.00327)$$

$$TTI_m = 1.30$$

This result means that during an average peak hour, the travel time through section #8 is 61% longer than the free-flow travel time. Once the mean TTI has been determined, it can be used to estimate other reliability performance measures. For example, the 95th-percentile TTI for section #8 can be calculated using the formula for an urban freeway:

$$TTI_{95} = 16.7754 \times \exp\left(-\frac{2.8221}{TTI_m}\right) = 16.7754 \times \exp\left(-\frac{2.8221}{1.30}\right) = 1.91$$

The policy 95th-percentile TTI (i.e., a TTI based on the posted speed limit) is then:

$$TTI_{p95} = TTI_{95} \times \frac{PSL}{FFS} = 1.91 \times \frac{60}{64.1} = 1.79$$

This result means that in 1 of every 20 peak hours on average, the travel time through section #8 is 79% longer than the travel time at the posted speed. The following table provides mean TTI, policy mean TTI, 95th-percentile TTI, and policy 95th-percentile TTI for each section along the study facility.

Section	1	2	3	4	5	6	7	8	9	10	11	12
TTI_m	1.00	1.00	1.00	1.02	1.00	1.00	1.00	1.30	1.01	1.09	1.27	1.04
TTI_{pm}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.22	1.00	1.02	1.18	1.00
TTI_{95}	1.00	1.00	1.00	1.07	1.00	1.00	1.00	1.91	1.02	1.27	1.80	1.12
TTI_{p95}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.79	1.00	1.19	1.69	1.05

Step 4. Determine Facility Reliability. This step aggregates the results from the individual sections to produce estimates of overall facility reliability. First, the travel times at the free-flow speed and the posted speed are determined for each section. Mean travel times (including incidents) are determined for each section by multiplying a section's free-flow travel time at the posted speed by its mean TTI. Similarly, 95th-percentile travel times are determined for each section by multiplying its free-flow travel time by its 95th-percentile TTI. The following table provides free-flow, posted speed, mean, and 95th-percentile travel times by section.

Section	1	2	3	4	5	6	7	8	9	10	11	12
T_{FFS}	16.9	15.7	15.2	53.9	14.0	9.5	11.2	71.9	15.7	15.2	56.7	18.0
T_{PSL}	18.0	16.8	16.2	57.6	15.0	10.2	12.0	76.8	16.8	16.2	60.6	19.2
T_m	16.9	15.7	15.2	55.2	14.0	9.5	11.2	93.5	15.8	16.6	71.8	18.8
T_{95}	16.9	15.7	15.2	57.4	14.0	9.5	11.2	137.6	16.0	19.2	102.3	20.2

Next, the individual section travel times at the posted speed are summed to produce the facility's travel time at the posted speed (335.4 s). Similarly, the individual section mean and 95th-percentile travel times are summed to produce the facility's mean (354.3 s) and 95th-percentile (435.5 s) travel times, respectively. The mean facility policy TTI is then the mean facility travel time divided by the travel time at the posted speed: $354.3 / 335.4 = 1.06$. The facility 95th-percentile policy TTI is the 95th-percentile travel time divided by the travel time at the posted speed: $435.5 / 335.4 = 1.30$. Thus, during 5% of peak hours, it takes 30% longer to travel the length of the facility (approximately 1 minute, 40 seconds longer) than the time required to travel at the posted speed.

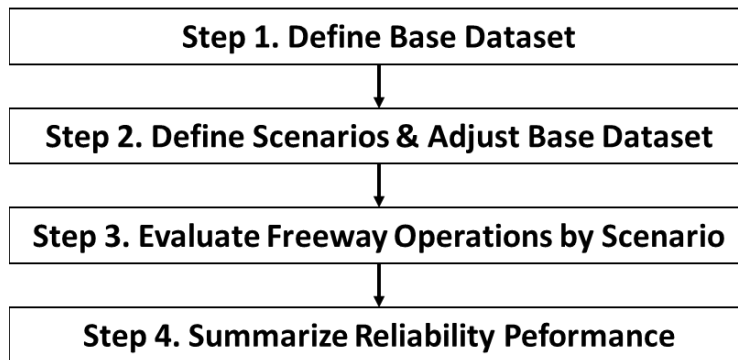
11.5.4 Detailed Analysis Method

Overview

The detailed analysis method uses the HCM 7th Edition freeway reliability analysis methodology described in HCM Chapters 11 and 25. The method predicts travel times for hundreds of scenarios over the course of a reporting period, which are subsequently assembled into a travel time distribution used to develop reliability performance measures of interest. The method can estimate the impacts of nonrecurring congestion

due to demand variability, weather, incidents, work zones, and special events (e.g., football games at UO or OSU, Bridge Pedal), and is sensitive to countermeasures that affect these factors (e.g., safety countermeasures that reduce a roadway's crash rate). The method can also be used to quantify the contribution of each type of congestion factor, as an aid to prioritizing measures to improve roadway reliability. The reliability reporting period is flexible and can incorporate either weekday (e.g., commuter route) or weekend (e.g., recreational route) study periods over the course of a month, a season (multiple months), or an entire year, depending on the analysis needs.

The detailed method involves the following steps:



This process requires software to implement because evaluating freeway operations by itself for one scenario is too computationally intensive to calculate by hand, let alone for hundreds of scenarios. Consequently, the remainder of the section describes the steps the analyst will need to follow to evaluate reliability, but not the specific calculations involved. For more details about the calculation process incorporated into software, see Chapters 11 and 25 of the HCM 7th Edition.

Calculation Process

Step 1: Define the Base Dataset

- **Step 1a. Define the study facility.**
 - The facility length should no greater than the distance a vehicle can travel in 15 minutes when the facility operates under capacity.
- **Step 1b. Define the study period length.**
 - The study period length can be between 1 and 24 hours. The minimum study period length should be sufficiently long to capture the formation and dissipation of queues.
- **Step 1c. Define the seed day.**
 - Fifteen-minute demand data for a single day (the “seed day”) will need to be provided for each analysis period within the study period. The specific day used is not particularly important, as long as demand on that day was unaffected by severe weather, incidents, work zones, or special events. Demand volumes for all other days within the reliability reporting period will be generated relative to the seed day. The seed day should be reported

as either the specific month and day when the demand data were collected, or as an “average day” if demands are generated by factoring an AADT.

- **Step 1d. Collect input data required for the freeway facilities core method.**
 - This process is summarized in Section 11.3. Required data include peak hour factors, truck percentages, terrain and area types, base free-flow speeds, lane and shoulder widths, and segment types (e.g., basic, merge, weaving) and lengths.

Step 2: Define Scenarios and Adjust the Base Dataset

- **Step 2a. Define the Reliability Reporting Period**
 - *Define the duration of the reliability reporting period (RRP).* The RRP is defined in Section 9.3.4. It typically covers a calendar year, but shorter timeframes may be appropriate for certain analyses (e.g., Memorial Day weekend to Labor Day weekend for routes connecting the Willamette Valley to the Oregon Coast). A minimum of month is recommended.
 - *Decide which days of the week to include in the RRP.* This decision will typically be driven by demand patterns on the highway and the specific analysis questions to be answered. For example, an urban freeway would typically evaluate weekdays, while a recreational route might focus on weekend days. A rural highway without strong commuting patterns might evaluate all days.
 - *Decide which days to exclude.* Depending on the needs of the analysis, non-typical days may be excluded from the RRP. For example, holidays could be excluded from an analysis of a commute route, but might be important to retain in an analysis of a recreational route.
- **Step 2b. Gather Reliability Inputs**
 - *Demand variability data.*
 - i. Option 1: Calculate demand adjustment factors (DAFs) for each day in the RRP using data from an Automatic Traffic Recorder (ATR) located on the study facility (see Appendix 11F). The DAF is equal to the demand on the given day divided by the demand on the seed day.
 - ii. Option 2: Apply Oregon default DAFs calculated for different facility types (see Appendix 11C).
 - *Weather data.*
 - i. Option 1: Calculate weather event probabilities using data from a weather station with a similar climate as the study facility (see Appendix 11F). Ten-year weather datasets are recommended to capture highly impactful weather events that do not occur every year.
 - ii. Option 2: Apply Oregon default weather event probabilities for the study facility (see Appendix 11C).
 - iii. Option 3: Apply the HCM default weather event probabilities for the Portland metropolitan area.

- iv. The analyst can optionally define DAFs associated with different types of severe weather events. No default DAFs are available, although extreme weather events are understood to affect traffic demands. Analyst judgment is required.
- *Incident data.*
 - i. Option 1: Calculate incident frequencies, severity distributions, and durations using local data (see Appendix 11F). Local data should only be used when incident logs are complete and accurate over the entire RRP.
 - ii. Option 2: Identify the crash rate for the study facility from ODOT's published crash rates (<https://www.oregon.gov/ODOT/Data/Pages/Crash.aspx>). Estimate an incident rate using the default Oregon incident-to-crash ratio and apply Oregon default incident severity distributions and durations (see Appendix 11C).
 - iii. The analyst can optionally define DAFs associated with different incident types. No default DAFs are available, although traffic management strategies to provide information about incidents can result in shifts in demand. Analyst judgment is required.
- *Short-term work zone and special event data.*
 - i. Local data must be used to account for short-term work zones and/or special events.
 - 1. Required work zone data include the schedule of days and times the work zone is in effect, start and end locations, reductions in the posted speed, lanes affected, and means of separating the work area from traffic. The analyst can optionally specify a demand adjustment associated with the work zone. It is recommended that long-term work zones be analyzed separately, with the freeway facility analysis (seed) file reflective of demand and roadway characteristics present during the work zone.
 - 2. Required special event data (e.g., football games, music festivals) include the schedule of days and times, duration, changes in demand, and changes in traffic control.
- **Step 2c. Define or Refine Global Inputs**
 - A well-calibrated seed file is preferred; changing global inputs is not recommended.
 - If necessary, facility-wide jam density and queue discharge capacity drop values can be calibrated, as described in Appendix 11B.
- **Step 2d. Define the Number of Scenario Replications for Reliability Analysis**
 - A minimum of 200 scenarios is recommended to generate a travel time distribution. This minimum number of scenarios allows relatively rare, but potentially highly impactful, weather and incident conditions to be modeled. Replications of an individual day (with different randomly

generated weather and incident events) are used to generate the variety of conditions that might be experienced during the RRP.

- The default number of replications for a 12-month RRP is four. The combination of five weekdays, 12 months, and four replications results in 240 scenarios.
 - Shorter RRP's will require a greater number of replications to produce a minimum 200 scenarios. Exhibit 11-9 in the HCM 7th Edition recommends the minimum number of replications to use for a given combination of number of months of the year and number of days in a week included in the RRP.
- **Step 2e. Assign Demand Variability by Day and Month to Scenarios**
 - The software applies the appropriate DAF determined in Step 2b to each scenario to account for differences in demand between the day represented by the scenario and the demand on the seed day.
 - **Step 2f. Assign Weather Effects to Scenarios**
 - The software randomly assigns weather events (weather type, starting time, and duration) to scenarios in software, based on their probability of occurrence on the day represented by the scenario.
 - The software applies capacity adjustment factors (CAFs) and speed adjustment factors (SAFs), and (if provided by the analyst) DAFs to account for the effects of severe weather on roadway operations during the analysis periods when the weather occurs.
 - **Step 2g. Assign Incidents to Scenarios**
 - The software randomly assigns incidents (incident severity, starting time, and duration) to scenarios in software, based on their probability of occurrence.
 - The software applies the capacity adjustment factors (CAFs) and speed adjustment factors (SAFs), and (if provided by the analyst) DAFs determined in Step 2b to account for the effects of the incident on roadway operations during the analysis periods affected by the incident.

- **Step 2h. Assign Work Zones to Scenarios**
 - If work zones have been specified by the analyst, the software will apply the CAFs, SAFs, and DAFs determined in Step 2b to the affected analysis periods to any scenario occurring on a day when a work zone is in effect.
- **Step 2i. Assign Special Events to Scenarios**
 - If special events have been specified by the analyst, the software will substitute the demands and roadway characteristics specified in Step 2b in place of the values used in the seed file. This substitution will occur for the affected analysis periods for any scenario occurring on a day when a special event occurs.

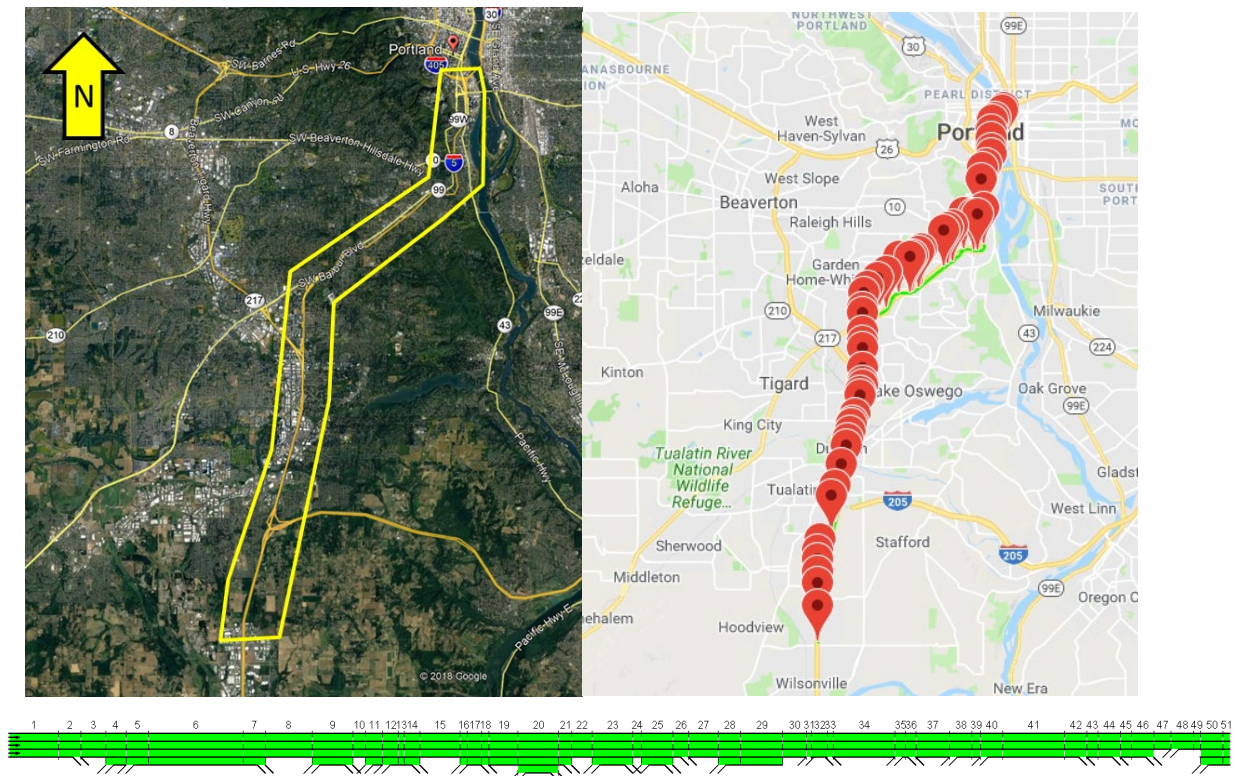
Step 3: Evaluate Freeway Operations by Scenario

- **Analyze Each Scenario**
 - The software performs the HCM freeway facilities core methodology (see Section 11.3.4) and determines the average facility travel time for each scenario, along with all other performance measures normally generated by the core methodology.

Step 4: Summarize Facility Performance

- **Step 4a. Assemble the Travel Time Distribution**
 - The software will compile the travel times associated with each scenario into a cumulative travel time distribution.
- **Step 4b. Compute Reliability Performance Measures**
 - The software will generate reliability performance measures from the travel time distribution. The specific measures automatically reported will vary by the software package used; however, the cumulative travel time distribution can be used to generate any other desired reliability measure not reported automatically. Typical reliability performance measures are described in Section 9.3.3. If the software does not generate policy TTIs (i.e., TTIs based on the posted speed limit rather than the free-flow speed), these can be calculated by multiplying the TTI by the ratio of the posted speed to the free-flow speed.
- **Step 4c. Report Reliability Performance Measures**
 - Only a selection from the range of available reliability performance measures should be reported and shared with decision-makers. Reporting too many measures could distract attention from key indicators. Section 9.3.3 provides recommendations on reliability performance measures to report.

Example 11-18 Freeway Reliability Analysis (Detailed Method)



Step 1: Define the Base Dataset

- **Step 1a. Define the study facility.**
 - Interstate I-5, MP 286 – 300
- **Step 1b. Define the study period length.**
 - 12:00 am – 11:59 pm (24 hours)
- **Step 1c. Define the seed day.**
 - 11/7/2017
- **Step 1d. Collect input data required for the freeway facilities core method.**
 - The geometry was created using FREEVAL's map-based segmentation interface, and demands were identified from 2017 AADT records. Oregon default capacities were applied for all merge, diverge, and weave segments.

Step 2: Define Scenarios and Adjust the Base Dataset

- **Step 2a. Define the Reliability Reporting Period**
 - Duration: 01/01/2017 – 12/31/2017
 - Days of the week: Monday–Friday
 - Excluded days: None

- **Step 2b. Gather Reliability Inputs**

- a. Demand variability data: Oregon default for Interstate - Urban (from Appendix 11C)

Month	Monday	Tuesday	Wednesday	Thursday	Friday
January	1.00	1.42	1.49	1.29	1.48
February	1.27	1.62	1.68	1.65	1.71
March	1.44	1.67	1.74	1.77	1.84
April	1.56	1.78	1.8	1.82	1.89
May	1.63	1.79	1.83	1.89	1.94
June	1.73	1.93	1.95	1.99	2.02
July	1.79	1.97	1.84	2.03	2.1
August	1.75	1.99	2.0	2.02	2.09
September	1.53	1.70	1.75	1.77	1.82
October	1.57	1.81	1.81	1.84	1.90
November	1.44	1.74	1.79	1.83	1.74
December	1.23	1.52	1.71	1.76	1.81

- o Weather data: HCM default for the Portland metropolitan area

Month	Medium Rain	Heavy Rain	Light Snow	Light/ Medium Snow	Medium/ Heavy Snow	Heavy Snow	Severe Cold	Low Visibility	Very Low Visibility	Minimum Visibility	Non-severe weather
January	1.2%	0.3%	1.0%	0.0%	0.1%	0.0%	0.0%	1.0%	0.0%	2.5%	93.9%
February	0.5%	0.1%	0.2%	0.0%	0.0%	0.0%	0.0%	0.6%	0.0%	2.3%	96.3%
March	0.8%	0.2%	0.1%	0.0%	0.0%	0.0%	0.0%	0.4%	0.0%	0.6%	97.8%
April	0.4%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%	0.3%	99.1%
May	0.5%	0.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.3%
June	0.5%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.4%
July	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.9%
August	0.1%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%	99.8%
September	0.3%	0.2%	0.0%	0.0%	0.0%	0.0%	0.0%	0.2%	0.0%	0.2%	99.1%
October	0.7%	0.1%	0.0%	0.0%	0.0%	0.0%	0.0%	0.5%	0.0%	1.3%	97.3%
November	1.2%	0.3%	1.0%	0.0%	0.1%	0.0%	0.0%	1.0%	0.0%	2.5%	93.9%
December	0.5%	0.1%	0.2%	0.0%	0.0%	0.0%	0.0%	0.6%	0.0%	2.3%	96.3%
Average Duration (Minutes)	37.9	18.3	112.6	20.5	21.9	6.6	0.0	37.4	0.0	131.2	37.9

- Incident data: The crash rate from ODOT tables varies by segment from 0.44 to 1.57 crashes per million vehicle miles. The default Oregon incident-to-crash ratio is 4.35.⁶ Other incident data are defaulted from values in Appendix 11C:

Incident Severity	Distribution %	Mean Duration (min)	Standard Deviation	Minimum Duration	Maximum Duration
Shoulder Closure	75.4	34.0	15.1	8.7	58.0
1-Lane Closure	19.6	34.6	13.8	16.0	58.2
2-Lane Closure	3.1	53.6	13.9	30.5	66.9
3-Lane Closure	1.9	67.9	21.9	36.0	93.3
4-Lane Closure	0.0	67.9	21.9	36.0	93.3

- Short-term work zones: None
- Special events: None

- **Step 2c. Define or Refine Global Inputs**

- No changes made.

- **Step 2d. Define the Number of Scenario Replications for Reliability Analysis**

- Number of months = 12
- Days per week = 5
- Number of replications = 4 (default)
- Check number of scenarios: $12 \times 5 \times 4 = 240 > 200$ (OK)

- **Step 2e. Assign Demand Variability by Day and Month to Scenarios**

- DAF relative to the seed date

Month	Monday	Tuesday	Wednesday	Thursday	Friday
January	0.575	0.816	0.856	0.741	0.851
February	0.730	0.931	0.966	0.948	0.983
March	0.828	0.960	1.000	1.017	1.057
April	0.897	1.023	1.034	1.046	1.086
May	0.937	1.029	1.052	1.086	1.115
June	0.994	1.109	1.121	1.144	1.161
July	1.029	1.132	1.057	1.167	1.207
August	1.006	1.144	1.149	1.161	1.201
September	0.879	0.977	1.006	1.017	1.046
October	0.902	1.040	1.040	1.057	1.092
November	0.828	1.000	1.029	1.052	1.000
December	0.707	0.874	0.983	1.011	1.040

⁶ Cambridge Systematics, Inc. Oregon SHRP2 C11 Reliability Analysis Implementation Plan, Task 2, Final Technical Memorandum #1. Medford, MA, Oct. 2018.

- **Step 2f. Assign Weather Effects to Scenarios**

- CAF, SAF, and DAF

	Medium Rain	Heavy Rain	Light Snow	Light/ Medium Snow	Medium/ Heavy Snow	Heavy Snow	Severe Cold	Low Visibility	Very Low Visibility	Minimum Visibility	Nor-severe Weather
CAF	0.93	0.86	0.96	0.91	0.89	0.78	0.92	0.90	0.88	0.90	1.00
SAF	0.93	0.92	0.87	0.86	0.84	0.83	0.93	0.94	0.92	0.92	1.00
DAF	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

- **Step 2g. Assign Incidents to Scenarios**

- SAFs and DAFs assigned default values of 1.00.
- CAFs taken from Oregon default values (Appendix 11C).

# of Lanes	Shoulder Closure	1-Lane Closure	2-Lane Closure	3-Lane Closure	4-Lane Closure
2	0.81	0.70			
3	0.83	0.74	0.51		
4	0.85	0.77	0.50	0.52	
5	0.87	0.81	0.67	0.50	0.50

- **Step 2h. Assign Work Zones to Scenarios**

- Skipped (no work zones identified)

- **Step 2i. Assign Special Events to Scenarios**

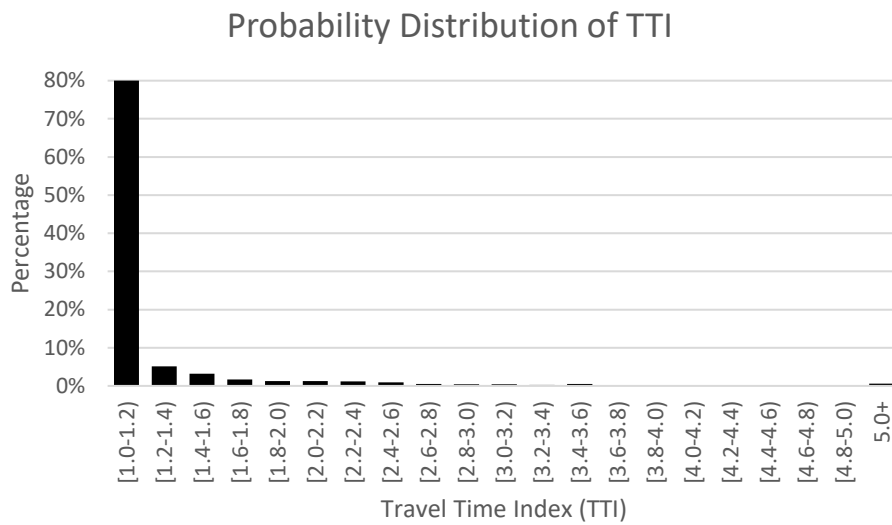
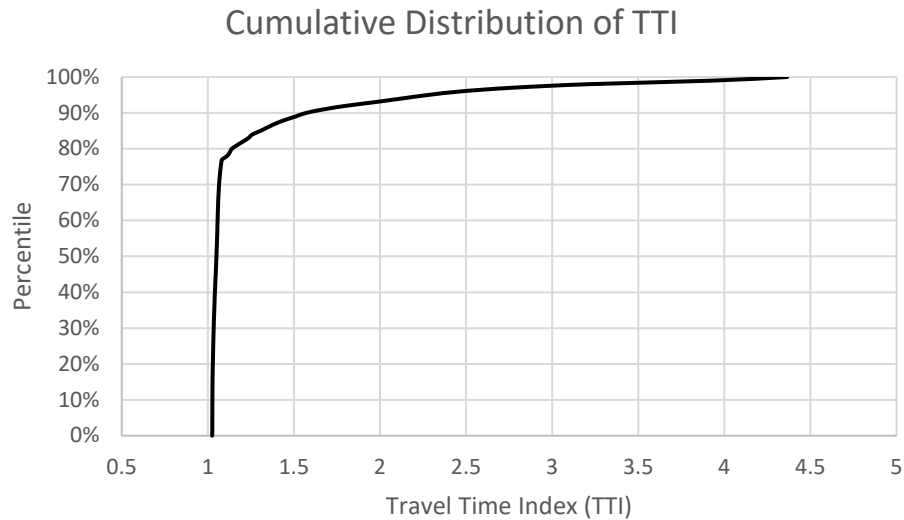
- Skipped (no special events identified)

Step 3: Evaluate Freeway Operations by Scenario

This step is performed in software.

Step 4: Summarize Facility Performance

- **Step 4a. Assemble the Travel Time Distribution**



- **Step 4b. Compute Reliability Performance Measures and Step 4c. Report Reliability Performance Measures**
 - All of these measures are reported by the software
 - TTI percentiles can be observed from the cumulative TTI graph
 - 50th-percentile TTI = 1.05
 - 80th-percentile TTI = 1.14
 - 95th-percentile TTI = 2.28
 - Mean TTI = 1.26
 - Misery index = average of the highest 5% of TTI values = 3.97
 - Reliability rating = % of TTIs at or below 1.33 = 85.57%
 - Vehicle miles of travel occurring at a TTI > 2 = 6.75%
-

11.5.5 TSMO Evaluation

Oregon DOT and other state agencies apply a range of Transportation Systems Management and Operations (TSMO) strategies to improve traffic operations on freeways. In the HCM, these strategies are commonly referred to as Active Travel and Demand Management (ATDM) strategies. Common TSMO/ATDM strategies for freeways include:

- Ramp Metering (static or dynamic),
- Part Time Shoulder Use (static or dynamic),
- Traveler information systems,
- Variable Speed Limit, and
- Managed Lanes.

The evaluation of TSMO strategies is possible as part of a detailed analysis methodology but is beyond the scope of the screening and sketch-planning methods. Within the detailed analysis method, there are two principal ways to conduct a TSMO strategy:

1. **Average TSMO performance** – evaluating the operational effects of the strategy on an average day using the core freeway facility method described in Section 11.3.4
2. **Reliability TSMO performance** – evaluating the effects of the strategy on whole-year performance using the detailed travel time reliability procedure described in Section 11.5.4.

While both analysis methods are viable, a reliability-based TSMO evaluation is generally preferred for the following reasons:

- TSMO strategies tend to impact reliability more than average-day performance (see Section 9.3),
- Strategies without capacity improvements won't impact performance of the average day,
- Several TSMO strategies are targeted at specific sources of unreliable travel (e.g. incident management or road weather management), and

- Several TSMO strategies respond dynamically to changing conditions (e.g. dynamic ramp metering or dynamic part time shoulder use).

In evaluating the viability of a specific TSMO strategy (or a combination of strategies), it is recommended to initially complete a reliability analysis using the method described in Section 11.5.4. Alternatively, probe-based travel time data for the subject facility can be used to explore variability of travel times, as well as underlying sources of unreliable travel. The TSMO strategy should be targeted to counteract specific causes of unreliable travel to maximize the return of investment. For example:

- A facility with unreliable winter travel may benefit from a road weather management system;
- A facility with high incident occurrence and long incident clearance times may benefit from safety improvements or a freeway service patrol system;
- A facility with high occurrence of tourist or special traffic resulting in demand fluctuations may benefit from traveler information systems, or
- A facility with unreliable travel due to fluctuations in on-ramp and mainline demand may benefit from a dynamic ramp metering or dynamic part-time shoulder use system.

The evaluation of a TSMO strategy in an HCM reliability context occurs through adjustments of the HCM calibration factors for free-flow speed (SAF), capacity (CAF), and demand (DAF). At the present time, limited guidance exists for these TSMO adjustment factors, which is summarized below.

Managed Lanes

The HCM 7th Edition provides procedures for evaluating freeways with managed lanes, also referred to as high-occupancy vehicle (HOV) lanes. The HCM managed lane method provides speed–flow relationships and maximum observed flow rates (not true capacities) for managed lane segments as a function of number of lanes (one vs. two lanes) and separation type (paint, buffer, or barrier). Managed lanes can further be evaluated in a facility context, as well as in a whole-year reliability analysis.

Part Time Shoulder Use

A part time shoulder use (PTSU) system results in the (time-limited) addition of an additional lane to selected segments. The shoulder lane has a typical capacity of approximately 1,600–1,800 passenger cars per hour per lane, based on FHWA guidance. Because the HCM method does not allow for lane-by-lane capacity values to be entered, a blended cross-section capacity is calculated following guidance in HCM Chapter 37:

$$AveCap(s) = \frac{CapShldr(s) + CapMFlanes(s) \times MFlanes(s)}{1 + MFlanes(s)}$$

where

$AveCap(s)$ = average capacity per lane for section s (veh/h/ln),
 $CapShldr(s)$ = capacity per shoulder lane for section s (veh/h/ln),
 $CapMFlanes(s)$ = capacity per mixed-flow lane in section s (veh/h/ln), and

$MFlanes(s)$ = number of mixed-flow lanes in section s (integer).

Ramp Metering

Ramp metering limits the allowable on-ramp demands to enter the freeway mainline, and is implemented through a ramp capacity limit in the HCM and FREEVAL. In addition, HCM Chapter 37 recommends the use of a capacity adjustment factor (CAF) of 1.03 to the freeway merge segment, to reflect reduced turbulence due to the ramp metering strategy. In addition, HCM Chapter 37 provides guidance for dynamic ramp metering evaluation.

Incident and Road Weather Management

An incident management program or freeway service patrol is implemented through a reduction in incident rate, incident severity, or incident duration. No default values are available for these adjustment, and local data or judgment should be used. Similarly, a road weather management system is likely to reduce the duration of weather impacts or weather-related incidents, which need to be obtained locally.

Traveler Information System

Traveler Information Systems primarily impact the demands on a facility. No defaults are available, but demand adjustment factors (DAFs) can be calibrated locally or entered based on assumptions (or objectives) of the specific strategy.

11.6 Truck Level of Service

[Reserved for future use.]

[Appendix 11A – Determining Free-Flow Speed](#)

[Appendix 11B – Freeway Facility Calibration](#)

[Appendix 11C – Oregon Default Values](#)

[Appendix 11D – Passenger Car Equivalents on Specific Grades](#)

[Appendix 11E – Software](#)

[Appendix 11F – Reliability Data Guidance](#)