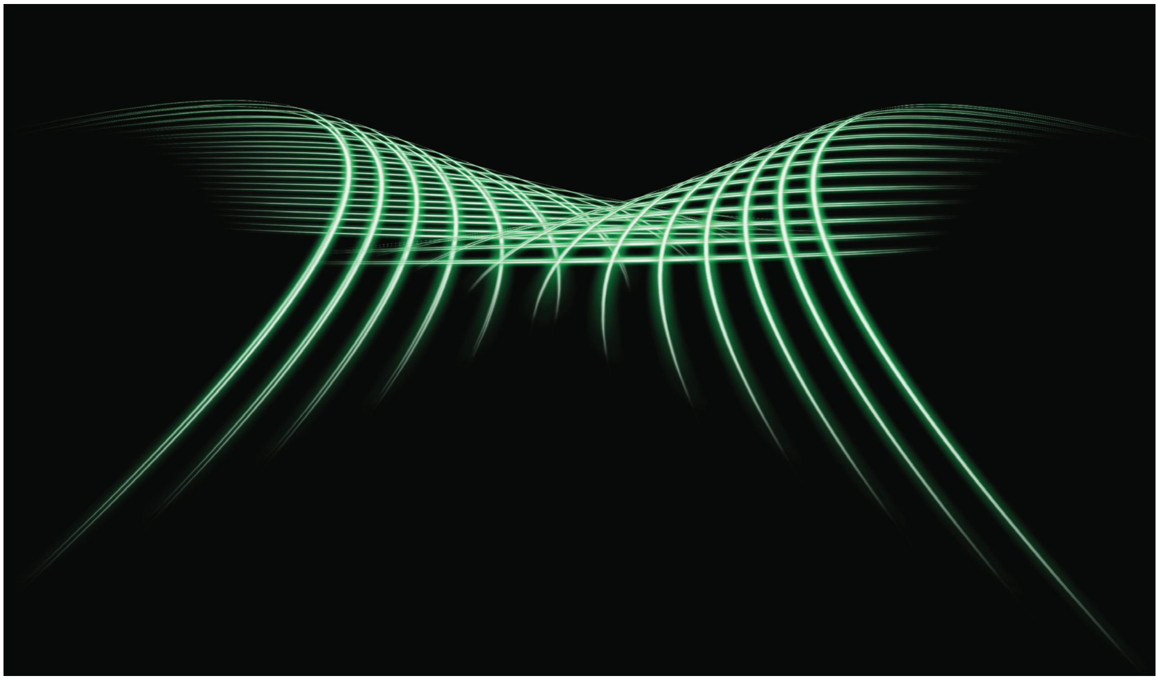
HCM2010

HIGHWAY CAPACITY MANUAL



CHAPTER 13

FREEWAY MERGE AND DIVERGE SEGMENTS

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HCM2010

HIGHWAY CAPACITY MANUAL

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CHAPTER 13
FREEWAY MERGE AND DIVERGE SEGMENTS

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Freeway merge and diverge segments occur primarily at on-ramp and off-ramp junctions with the freeway mainline. They can also occur at major merge or diverge points where mainline roadways join or separate.

Freeway merge and diverge segments include ramp junctions and points where mainline roadways join or separate.

A ramp is a dedicated roadway providing a connection between two highway facilities. On freeways, all movements onto and off of the freeway are made at ramp junctions—designed to permit relatively high-speed merging and diverging maneuvers while limiting the disruption to the main traffic stream. Some ramps on freeways connect to collector–distributor (C-D) roadways, which in turn provide a junction with the freeway mainline. Ramps may appear on multilane highways, two-lane highways, arterials, and urban streets, but such facilities may also use signalized and unsignalized intersections at such junctions.

The procedures in **Chapter 13, Freeway Merge and Diverge Segments,** focus on ramp–freeway junctions, but guidance is also provided to allow approximate use of such procedures on multilane highways and on C-D roadways.

This chapter provides guidance for using the procedures on multilane highways and C-D roadways.

RAMP COMPONENTS

A ramp consists of three elements: the ramp roadway and two junctions. Junctions vary greatly in design and control features but generally fit into one of these categories:

- Ramp–freeway junctions (or a junction with a C-D roadway or multilane highway segment), or
- Ramp–street junctions.

When a ramp connects one freeway to another, the ramp consists of two ramp–freeway junctions and the ramp roadway. When a ramp connects a freeway to a surface facility, it generally consists of a ramp–freeway junction, the ramp roadway, and a ramp–street junction. When a ramp connection to a surface facility (such as a multilane highway) or a C-D roadway is designed for high-speed merging or diverging without control, it may be classified as a ramp–freeway junction for the purpose of analysis.

Ramps to multilane highways and C-D roadways that are designed for high-speed merging or diverging may be classified as ramp–freeway junctions for analysis purposes.

Ramp–street junctions may be uncontrolled, STOP-controlled, YIELD-controlled, or signalized. Analysis of ramp–street junctions is not detailed in this chapter; rather, it is discussed in Chapter 22, Interchange Ramp Terminals. Note, however, that an off-ramp–street junction, particularly if signalized, can result in queuing on the ramp roadway that can influence operations at the ramp–freeway junction and even mainline freeway conditions. Mainline operations can also be affected by platoon entries created by ramp–street intersection control.

See Chapter 22 for a discussion of ramp–street junctions.

Ramp queuing from a junction of an off-ramp and street can influence the operations of the ramp–freeway junction and the upstream freeway.

The geometric characteristics of ramp–freeway junctions vary. The length and type (parallel, taper) of acceleration or deceleration lane(s), the free-flow speed (FFS) of both the ramp and the freeway in the vicinity of the ramp, proximity of other ramps, and other elements all affect merging and diverging operations.

CLASSIFICATION OF RAMPS

Ramps and ramp–freeway junctions may occur in a wide variety of configurations. Some of the key characteristics of ramps and ramp junctions are summarized below:

- Ramp–freeway junctions that accommodate merging maneuvers are classified as *on-ramps.* Those that accommodate diverging maneuvers are classified as *off-ramps.* Where the junctions accommodate the merging of two major facilities, they are classified as *major merge* junctions. Where they accommodate the divergence of two major roadways, they are classified as *major diverge* junctions.
- The majority of ramps are right-hand ramps. Some, however, join with the left lane(s) of the freeway and are classified as left-hand ramps.
- Ramp roadways may have one or two lanes. At on-ramp freeway junctions, most two-lane ramp roadways merge into a single lane before merging with the freeway. In this case, the junction is classified as a one-lane ramp–freeway junction on the basis of the methodology of this chapter. In other cases, a two-lane ramp–freeway merge exists, and a special analysis model is used (see this chapter's Special Cases section).
- For two-lane off-ramps, a single lane may exist at the ramp–freeway diverge, with the roadway widening to two lanes after the diverge. As with on-ramps, such cases are classified as one-lane ramp–freeway junctions on the basis of this chapter's methodology. Two-lane off-ramp roadways, however, often have two lanes at the diverge point as well. These are treated by using a special model (see this chapter's Special Cases section).
- Ramp–freeway merge and diverge operations are affected by the size of the freeway segment (in one direction).
- Ramp–freeway merge and diverge operations may be affected by the proximity of adjacent ramps and the flow rates on those ramps.

Left-hand ramps are considered as special cases later in this chapter.

Merge and diverge segments with two lanes at the point of merge or diverge are considered as special cases later in this chapter.

The number of combinations of these characteristics that can occur is very large. For any analysis, all of these (and other) characteristics must be specified if meaningful results are to be obtained.

RAMP AND RAMP JUNCTION ANALYSIS BOUNDARIES

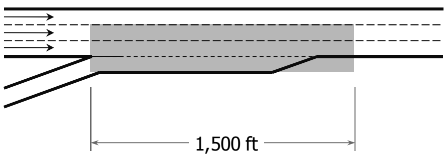
Ramps and ramp junctions do not operate independently of the roadways they connect. Thus, operating conditions on the main roadways can affect operations on the ramp and ramp junctions, and vice versa. In particular, a breakdown [Level of Service (LOS) F] at a ramp–freeway junction may have serious effects on the freeway upstream or downstream of the junction. These effects can influence freeway operations for miles in the worst cases.

With undersaturated conditions, the operational impacts of ramp–freeway junctions occur within a 1,500-ft-long influence area.

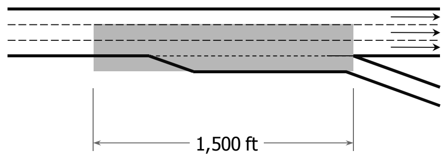
For most stable operations, however, studies (*1*) have shown that the operational impacts of ramp–freeway junctions are more localized. Thus, the methodology presented in this chapter predicts the operating characteristics within a defined ramp influence area. For right-hand on-ramps, the ramp influence area includes the acceleration lane(s) and Lanes 1 and 2 of the freeway mainline (rightmost and second rightmost) for a distance of 1,500 ft downstream of the merge point. For right-hand off-ramps, the ramp influence area includes the deceleration lane(s) and Lanes 1 and 2 of the freeway for a distance of 1,500 ft upstream of the diverge point. Exhibit 13-1 illustrates the definition of ramp influence areas. For left-hand ramps, the two leftmost lanes of the freeway are affected.

The influence area includes the acceleration/deceleration lane and the right two lanes of the freeway (left two lanes for left-hand ramps).

Exhibit 13-1
Ramp Influence Areas Illustrated



(a) Merge Influence Area



(b) Diverge Influence Area

RAMP–FREEWAY JUNCTION OPERATIONAL CONDITIONS

Ramp–freeway junctions create turbulence in the merging or diverging traffic stream. In general, the turbulence is the result of high lane-changing rates.

The action of individual merging vehicles entering the Lane 1 traffic stream creates turbulence in the vicinity of the ramp. Approaching freeway vehicles move toward the left to avoid the turbulence. Thus, the ramp influence area experiences a higher rate of lane-changing than is normally present on ramp-free portions of freeway.

Ramp influence areas experience higher rates of lane-changing than normally occur in basic freeway segments.

At off-ramps, the basic maneuver is a diverge—a single traffic stream separating into two streams. Exiting vehicles must occupy the lane(s) adjacent to the off-ramp (Lane 1 for a single-lane right-hand off-ramp). Thus, as the off-ramp is approached, vehicles leaving the freeway must move to the right. This causes other freeway vehicles to redistribute as they move left to avoid the turbulence of the immediate diverge area. Again, the ramp influence area has a higher rate of lane-changing than is normally present on ramp-free portions of freeway.

Vehicle interactions are dynamic in ramp influence areas. Approaching freeway through vehicles will move left as long as there is capacity to do so. Whereas the intensity of ramp flow influences the behavior of through freeway vehicles, general freeway congestion can also act to limit ramp flow, causing diversion to other interchanges or routes.

Exhibit 13-1 and the accompanying discussion relate to single-lane right-hand ramps. For two-lane right-hand ramps, the characteristics are basically the same, except that two acceleration or deceleration lanes may be present. For left-hand ramps, merging and diverging obviously take place on the left side of the freeway. This chapter's methodology is based on right-hand ramps, but modifications allowing the adaptation of the methodology to consider left-hand ramps are presented in the Special Cases section of this chapter.

BASE CONDITIONS

The base conditions for the methodology presented in this chapter are the same as for other types of freeway segments:

Base conditions for merge and diverge segments are the same as for other types of freeway segments.

- No heavy vehicles,
- 12-ft lanes,
- Adequate lateral clearances (≥6 ft), and
- Road users familiar with the facility (i.e., $f_p = 1.00$).

LOS CRITERIA FOR MERGE AND DIVERGE SEGMENTS

LOS A–E is defined in terms of density; LOS F exists when demand exceeds capacity.

Merge/diverge segment LOS is defined in terms of density for all cases of stable operation (LOS A–E). LOS F exists when the freeway demand exceeds the capacity of the upstream (diverges) or downstream (merges) freeway segment, or where the off-ramp demand exceeds the off-ramp capacity.

At LOS A, unrestricted operations exist, and the density is low enough to permit smooth merging or diverging with very little turbulence in the traffic stream. At LOS B, merging and diverging maneuvers become noticeable to through drivers, and minimal turbulence occurs. At LOS C, speed within the ramp influence area begins to decline as turbulence levels become much more noticeable. Both ramp and freeway vehicles begin to adjust their speeds to accomplish smooth transitions. At LOS D, turbulence levels in the influence area become intrusive, and virtually all vehicles slow to accommodate merging or diverging maneuvers. Some ramp queues may form at heavily used on-ramps, but freeway operation remains stable. LOS E represents operating conditions approaching or at capacity. Small changes in demand or disruptions within the traffic stream can cause both ramp and freeway queues to form.

LOS F defines operating conditions within queues that form on both the ramp and the freeway mainline when capacity is exceeded by demand. For on-ramps, LOS F exists when the total demand flow rate from the upstream freeway segment and the on-ramp exceeds the capacity of the downstream freeway segment. For off-ramps, LOS F exists when the total demand flow rate on the approaching upstream freeway segment exceeds the capacity of the upstream freeway segment. LOS F also occurs when the off-ramp demand exceeds the capacity of the off-ramp.

Exhibit 13-2 summarizes the LOS criteria for freeway merge and diverge segments. These criteria apply to all ramp–freeway junctions and may also be applied to major merges and diverges; high-speed, uncontrolled merge or diverge ramps on multilane highway sections; and merges and diverges on freeway C-D roadways. LOS is not defined for ramp roadways, while the LOS of a ramp–street junction is defined in Chapter 22, Interchange Ramp Terminals.

Exhibit 13-2
LOS Criteria for Freeway Merge and Diverge Segments

LOS	Density (pc/mi/ln)	Comments
A	≤10	Unrestricted operations
B	>10–20	Merging and diverging maneuvers noticeable to drivers
C	>20–28	Influence area speeds begin to decline
D	>28–35	Influence area turbulence becomes intrusive
E	>35	Turbulence felt by virtually all drivers
F	Demand exceeds capacity	Ramp and freeway queues form

REQUIRED INPUT DATA

The analysis of a ramp–freeway junction requires details concerning the junction under analysis and adjacent upstream and downstream ramps, in addition to the data required for a typical freeway analysis.

Data Describing the Freeway

The following information concerning the freeway mainline is needed to conduct an analysis:

1. FFS: 55–75 mi/h;
2. Number of mainline freeway lanes: 2–5;
3. Terrain: level, rolling, or mountainous; or percent grade and length;
4. Heavy vehicle presence: percent trucks and buses, percent recreational vehicles (RVs);
5. Demand flow rate immediately upstream of the ramp–freeway junction;
6. Peak hour factor: up to 1.00; and
7. Driver population factor: 0.85–1.00.

FFS is best measured in the field but can be estimated by using the methodology for basic freeway segments or multilane highways, as applicable.

The freeway FFS is best measured in the field. If a field measurement is not available, one may be estimated by using the methodology for basic freeway segments presented in Chapter 11, Basic Freeway Segments. To use this methodology, information on lane widths, lateral clearances, number of lanes, and total ramp density is required. If the ramp junction is located on a multilane highway or C-D roadway, the FFS range is somewhat lower (45–60 mi/h) and can be estimated by using the methodology in Chapter 14, Multilane Highways, if no field measurements are available. The methodology can be applied to facilities with any FFS. Its use with multilane highways or C-D roadways must be considered approximate, however, since it was not calibrated with data from these types of facilities.

Where the ramp–freeway junction is on a specific grade, the length of the grade is measured from its beginning to the point of the ramp junction.

The driver population factor is generally 1.00, unless the demand consists primarily of drivers who are not regular users of the facility. In such cases, an appropriate value should be based on field observations at the location under study or at similar nearby locations.

Data Describing the Ramp–Freeway Junction

The following information concerning the ramp–freeway junction is needed to conduct an analysis:

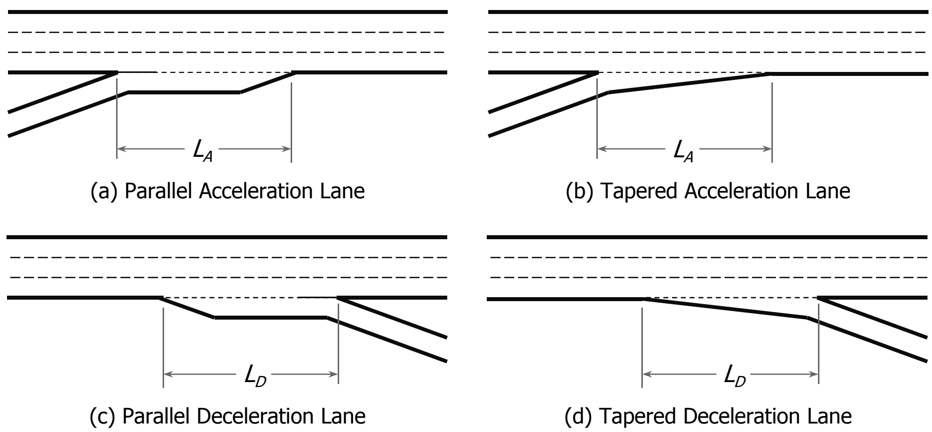
1. Type of ramp: on-ramp, off-ramp, major merge, major diverge;
2. Side of junction: right-hand, left-hand;
3. Number of lanes on ramp roadway: 1 lane, 2 lanes;
4. Number of ramp lanes at ramp–freeway junction: 1 lane, 2 lanes;
5. Length of acceleration/deceleration lane(s);
6. FFS of the ramp roadway: 20–50 mi/h;

7. Ramp terrain: level, rolling, or mountainous; or percent grade, length;
8. Demand flow rate on ramp;
9. Heavy vehicle presence: percent trucks and buses, percent RVs;
10. Peak hour factor: up to 1.00;
11. Driver population factor: 0.85–1.00; and
12. For adjacent upstream or downstream ramps:
    a. Upstream or downstream distance to the merge/diverge under study,
    b. Demand flow rate on the upstream or downstream ramp, and
    c. Peak hour factor and heavy vehicle percentages for the upstream or downstream ramp.

The length of the acceleration or deceleration lane includes the tapered portion of the ramp.

The length of the acceleration or deceleration lane includes the tapered portion of the ramp. Exhibit 13-3 illustrates lengths for both parallel and tapered ramp designs.

Exhibit 13-3
Measuring the Length of Acceleration and Deceleration Lanes



Source: *Traffic Engineering,* 3rd edition (*2*).

Length of Analysis Period

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

2. METHODOLOGY

SCOPE OF THE METHODOLOGY

This chapter focuses on the operation of ramp–freeway junctions. The procedures may be applied in an approximate manner to completely uncontrolled ramp terminals on other types of facilities, such as multilane highways, two-lane highways, and freeway C-D roadways that are part of interchanges.

This chapter's procedures can be used to identify likely congestion at ramp–freeway junctions (LOS F) and to analyze undersaturated operations (LOS A–E) at ramp–freeway junctions. Chapter 10, Freeway Facilities, provides procedures for a more detailed analysis of oversaturated flow and congested conditions along a freeway section, including weaving, merge and diverge, and basic freeway segments.

The procedures in this chapter result primarily from studies conducted under National Cooperative Highway Research Program Project 3-37 (*1*, *2*). Some special applications resulted from adaptations of procedures developed in the 1970s (*3*). American Association of State Highway and Transportation Officials policies (*4*) contain additional material on the geometric design and design criteria for ramps.

LIMITATIONS OF THE METHODOLOGY

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

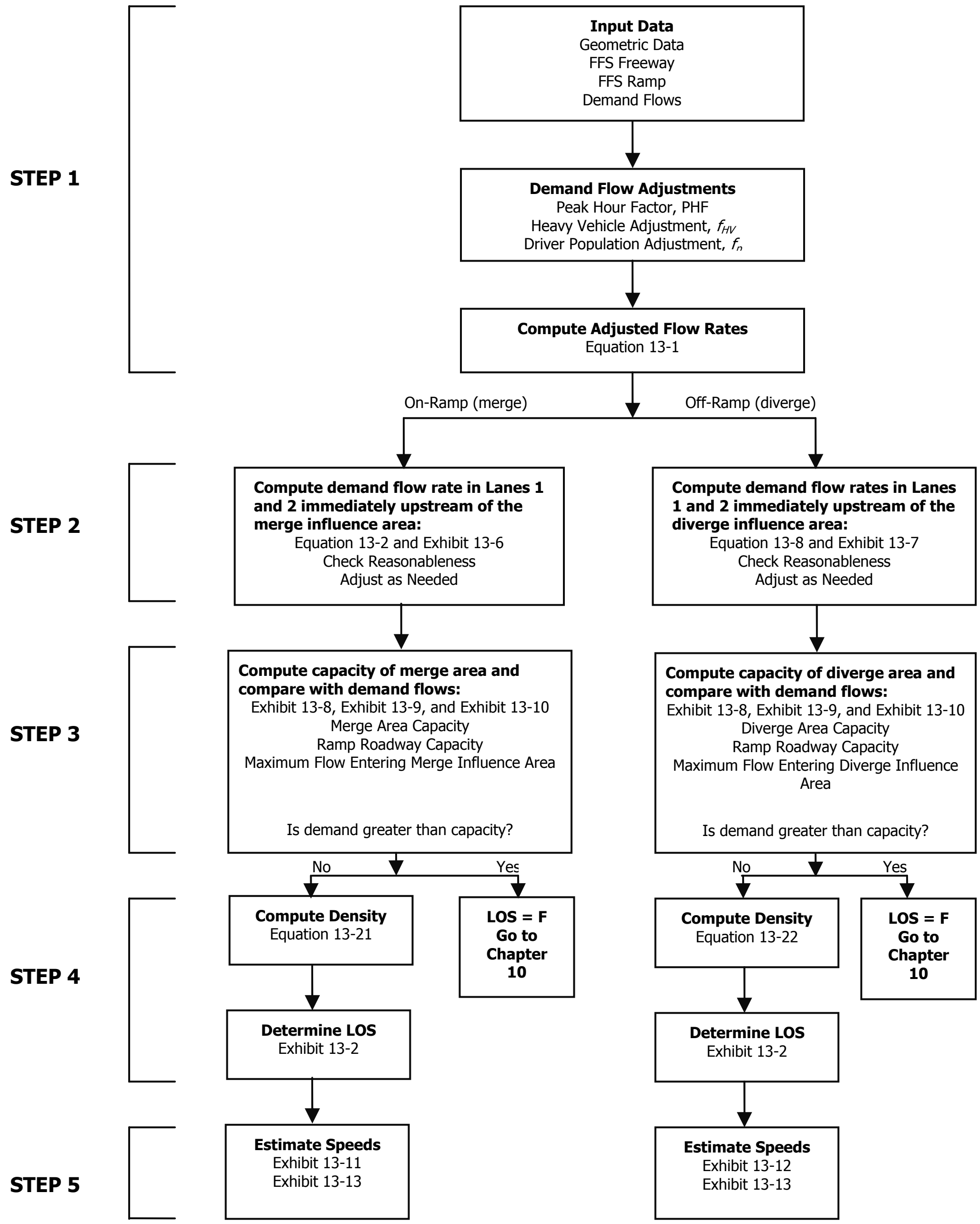
- Special lanes, such as high-occupancy vehicle (HOV) lanes, as ramp entry lanes;
- Ramp metering; or
- Intelligent transportation system features.

The methodology does not explicitly take into account posted speed limits or level of police enforcement. In some cases, low speed limits and strict enforcement could result in lower speeds and higher densities than those anticipated by this methodology.

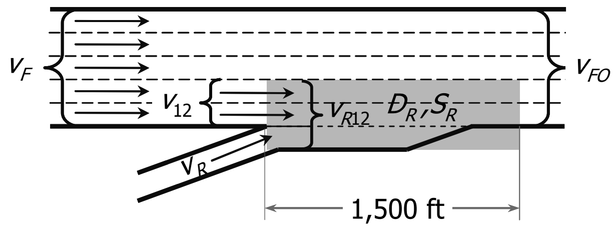
OVERVIEW

Exhibit 13-4 illustrates the computational methodology applied to the analysis of ramp–freeway junctions. The analysis is generally entered with known geometric and demand factors. The primary outputs of the analysis are LOS and capacity. The methodology estimates the density and speed in the ramp influence area.

Exhibit 13-4
Flowchart for Analysis of Ramp–Freeway Junctions



As previously discussed, the methodology focuses on modeling the operating conditions within the ramp influence area, as defined in Exhibit 13-1. Because the ramp influence area includes only Lanes 1 and 2 of the freeway, an important part of the methodology involves predicting the number of approaching freeway vehicles that remain in these lanes immediately upstream of the ramp–freeway junction. While operations in other freeway lanes may be affected by merging and diverging maneuvers, particularly under heavy flow, the defined influence area experiences most of the operational impacts across all levels of service (except LOS F). At breakdown, queues and operational impacts may extend well beyond the defined influence area. Exhibit 13-5 illustrates key variables involved in the methodology.



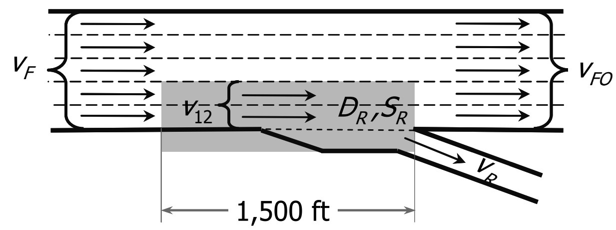


Exhibit 13-5
Key Ramp Junction Variables

The variables illustrated in Exhibit 13-5 are defined as follows:

v_F = flow rate on freeway immediately upstream of the ramp influence area under study (pc/h),

v_{12} = flow rate in freeway Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h),

v_{FO} = flow rate on the freeway immediately downstream of the merge or diverge area (pc/h),

v_R = flow rate on the on-ramp or off-ramp (pc/h),

v_{R12} = sum of the flow rates in Lanes 1 and 2 and the ramp flow rate (on-ramps only) (pc/h),

D_R = density in the ramp influence area (pc/mi/ln), and

S_R = average speed in the ramp influence area (mi/h).

The computational process illustrated in Exhibit 13-4 may be broken into five primary steps:

1. Specifying input variables and converting demand volumes to demand flow rates in passenger cars per hour under equivalent base conditions;
2. Estimating the flow remaining in Lanes 1 and 2 of the freeway immediately upstream of the merge or diverge influence area;
3. Estimating the capacity of the merge or diverge area and comparing the capacity with the converted demand flow rates;
4. For stable operations (i.e., demand is less than or equal to capacity), estimating the density within the ramp influence area and determining the expected LOS; and
5. When desired, estimating the average speed of vehicles within the ramp influence area.

Each step is discussed in detail in the sections that follow.

COMPUTATIONAL STEPS

The methodology was calibrated for one-lane, right-side ramp–freeway junctions. Other situations are addressed in the Special Cases section.

The methodology described in this section was calibrated for one-lane, right-side ramp–freeway junctions. All other cases—two-lane ramp junctions, left-side ramps, and major merge and diverge configurations—are analyzed with the modified procedures detailed in the Special Cases section.

Step 1: Specify Inputs and Convert Demand Volumes to Demand Flow Rates

All geometric and traffic variables for the ramp–freeway junction should be specified as inputs to the methodology, as discussed previously. Flow rates on the approaching freeway, on the ramp, and on any existing upstream or downstream adjacent ramps must be converted from hourly volumes (in vehicles per hour) to peak 15-min flow rates (in passenger cars per hour) under equivalent ideal conditions:

Equation 13-1

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

where

v_i = demand flow rate for movement *i* (pc/h),

V_i = demand volume for movement *i* (veh/h),

PHF = peak hour factor,

f_{HV} = adjustment factor for heavy vehicle presence, and

f_p = adjustment factor for driver population.

If demand data or forecasts are already stated as 15-min flow rates, *PHF* is set at 1.00. Adjustment factors are the same as those used in Chapter 11, Basic Freeway Segments. These can also be used when the primary facility is a multilane highway or a C-D roadway in a freeway interchange.

Step 2: Estimate the Approaching Flow Rate in Lanes 1 and 2 of the Freeway Immediately Upstream of the Ramp Influence Area

Because the ramp influence area includes Lanes 1 and 2 of the freeway (for a right-hand ramp), a critical step in the analysis is estimating the total flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area.

The distribution of freeway vehicles approaching a ramp influence area is affected by a number of variables:

- Total freeway flow approaching the ramp influence area v_F (pc/h),
- Total on- or off-ramp flow v_R (pc/h),
- Total length of the acceleration lane L_A or deceleration lane L_D (ft), and
- FFS of the ramp at the junction point S_{FR} (mi/h).

The lane distribution of approaching freeway vehicles may also be affected by adjacent upstream or downstream ramps. Nearby ramps can influence lane distribution as drivers execute lane changes to position themselves for ramp movements at adjacent ramps. An on-ramp, for example, located only a few hundred feet upstream of a subject ramp may result in additional vehicles in Lanes 1 and 2 at the subject ramp. A downstream off-ramp near a subject ramp may contain additional vehicles in Lanes 1 and 2 destined for the downstream ramp.

Theoretically, the influence of adjacent upstream and downstream ramps does not depend on the size of the freeway. In practical terms, however, this methodology only accounts for such influences on six-lane freeways (three lanes in one direction). On four-lane freeways (two lanes in one direction), the determination of v_{12} is trivial: since only Lanes 1 and 2 exist, all approaching freeway vehicles are, by definition, in Lanes 1 and 2 regardless of the proximity of adjacent ramps. On eight-lane (four lanes in one direction) or larger freeways, the data are insufficient to determine the impact of adjacent ramps on lane distribution. In addition, two-lane ramps are never included as "adjacent" ramps under these procedures.

For six-lane freeways, the methodology includes a process for determining whether adjacent upstream and downstream ramps are close enough to influence lane distribution at a subject ramp junction. When such ramps are close enough, the following additional variables may be involved:

- Flow rate on the adjacent upstream ramp v_U (pc/h),
- Distance between the subject ramp junction and the adjacent upstream ramp junction L_{UP} (ft),
- Flow rate on the adjacent downstream ramp v_D (pc/h), and
- Distance between the subject ramp junction and the adjacent downstream ramp junction L_{DOWN} (ft).

The distance to adjacent ramps is measured between the points at which the left edge of the leftmost ramp lane meets the right-lane edge of the freeway.

In practical terms, the influence of adjacent ramps rarely extends more than approximately 8,000 ft. Nevertheless, whether an adjacent ramp on a six-lane freeway has influence should be determined by using the algorithms specified in this methodology.

Of all these variables, the total approaching freeway flow has the greatest impact on flow in Lanes 1 and 2. The models are structured to account for this phenomenon without distorting other relationships. Longer acceleration and deceleration lanes lessen turbulence as ramp vehicles enter or leave the freeway. This leads to lower densities and higher speeds in the ramp influence area. When the ramp has a higher FFS, vehicles can enter and leave the freeway at higher speeds, and approaching freeway vehicles tend to move left to avoid the possibility of high-speed turbulence. This produces greater presegregation and smoother flow across all freeway lanes.

While the models are similarly structured, there are distinct differences between the lane distribution impacts of on-ramps and off-ramps. Separate models are presented for each case in the sections that follow.

Estimating Flow in Lanes 1 and 2 for On-Ramps (Merge Areas)

The general model for on-ramps specifies that flow in Lanes 1 and 2 immediately upstream of the merge influence area is simply a proportion of the approaching freeway flow, as shown in Equation 13-2:

Equation 13-2

$$v_{12} = v_F \times P_{FM}$$

where

v_{12} = flow rate in Lanes 1 and 2 (pc/h),

v_F = total flow rate on freeway immediately upstream of the on-ramp (merge) influence area (pc/h), and

P_{FM} = proportion of freeway vehicles remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area.

Exhibit 13-6 shows the algorithms used to determine P_{FM} for on-ramps or merge areas. All variables in Exhibit 13-6 are as previously defined.

Three equations are provided for six-lane freeways. Equation 13-3 is the base case for isolated ramps and for cases in which adjacent ramps are not found to influence merging operations. Equation 13-4 addresses cases with an upstream adjacent off-ramp, while Equation 13-5 addresses cases with a downstream adjacent off-ramp. Adjacent on-ramps (either upstream or downstream) have not been found to have a statistically significant impact on operations and are therefore ignored; Equation 13-3 is applied in such cases.

Adjacent upstream or downstream ramps do not affect the prediction of v_{12} for two-lane (one direction) freeway segments, since *all* vehicles are in Lanes 1 and 2. Data have been insufficient to determine whether adjacent ramps influence lane distribution on four-lane (one direction) freeway segments, and thus no such impact is used in this methodology.

Where an upstream or downstream adjacent off-ramp exists on a six-lane freeway, a determination as to whether the ramp is close enough to the subject merge area to influence the area's operation is necessary. The determination is made by finding the equilibrium separation distance L_{EQ}. If the actual distance is larger than or equal to L_{EQ}, Equation 13-3 should be used. If the actual distance is shorter than L_{EQ}, then Equation 13-4 or Equation 13-5 should be used as appropriate.

Exhibit 13-6
Models for Predicting P_{FM} at On-Ramps or Merge Areas

No. of Freeway Lanes[a]	Model(s) for Determining P_{FM}		
4	$P_{FM} = 1.000$		
6	$P_{FM} = 0.5775 + 0.000028\ L_A$		Equation 13-3
	$P_{FM} = 0.7289 - 0.0000135\ (v_F + v_R) - 0.003296 S_{FR} + 0.000063 L_{\mathrm{UP}}$		Equation 13-4
	$P_{FM} = 0.5487 + 0.2628\ (v_D / L_{\mathrm{DOWN}})$		Equation 13-5
8	For $v_F / S_{FR} \le 72$: $P_{FM} = 0.2178 - 0.000125 v_R + 0.01115\ (L_A / S_{FR})$		
	For $v_F / S_{FR} > 72$: $P_{FM} = 0.2178 - 0.000125 v_R$		

SELECTING EQUATIONS FOR P_{FM} FOR SIX-LANE FREEWAYS

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	On	None	Equation 13-3
None	On	On	Equation 13-3
None	On	Off	Equation 13-5 or 13-3
On	On	None	Equation 13-3
Off	On	None	Equation 13-4 or 13-3
On	On	On	Equation 13-3
On	On	Off	Equation 13-5 or 13-3
Off	On	On	Equation 13-4 or 13-3
Off	On	Off	Equation 13-5 or 13-4 or 13-3

Note: [a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
If an adjacent diverge on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 13-3.

The equilibrium distance is obtained by finding the distance at which Equation 13-3 would yield the same value of P_{FM} as Equation 13-4 or Equation 13-5, as appropriate. This results in the following:

For adjacent upstream off-ramps, use Equation 13-6:

$$L_{EQ} = 0.214(v_F + v_R) + 0.444 L_A + 52.32 S_{FR} - 2{,}403$$

Equation 13-6

For adjacent downstream off-ramps, use Equation 13-7:

$$L_{EQ} = \frac{v_D}{0.1096 + 0.000107 L_A}$$

Equation 13-7

where all terms are as previously defined.

When both adjacent upstream and downstream off-ramps are present, the larger resulting value of P_{FM} is used.

A special case exists when both an upstream and a downstream adjacent off-ramp are present. In such cases, two different values of P_{FM} could arise: one from consideration of the upstream ramp and the other from consideration of the downstream ramp (they cannot be considered simultaneously). In such cases, the analysis resulting in the larger value of P_{FM} is used.

When an adjacent off-ramp to a merge area on a six-lane freeway is not a one-lane, right-side off-ramp, apply Equation 13-3.

In addition, the algorithms used to include the impact of an upstream or downstream off-ramp on a six-lane freeway are only valid for single-lane, right-side adjacent ramps. Where adjacent off-ramps consist of two-lane junctions or major diverge configurations, or where they are on the left side of the freeway, Equation 13-3 is always applied.

Estimating Flow in Lanes 1 and 2 for Off-Ramps (Diverge Areas)

When approaching an off-ramp (diverge area), all off-ramp traffic must be in freeway Lanes 1 and 2 immediately upstream of the ramp to execute the desired maneuver. Thus, for off-ramps, the flow in Lanes 1 and 2 consists of all off-ramp vehicles and a proportion of freeway through vehicles, as in Equation 13-8:

Equation 13-8

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

where

v_{12} = flow rate in Lanes 1 and 2 of the freeway immediately upstream of the deceleration lane (pc/h),

v_R = flow rate on the off-ramp (pc/h), and

P_{FD} = proportion of diverging traffic remaining in Lanes 1 and 2 immediately upstream of the deceleration lane.

For off-ramps, the point at which flows are defined is the beginning of the deceleration lane(s), regardless of whether this point is within or outside the ramp influence area.

Exhibit 13-7 contains the equations used to estimate P_{FD} at off-ramp diverge areas. As was the case for on-ramps (merge areas), the value of P_{FD} for four-lane freeways is trivial, since only Lanes 1 and 2 exist.

Exhibit 13-7
Models for Predicting P_{FD} at Off-Ramps or Diverge Areas

No. of Freeway Lanes[a]	Model(s) for Determining P_{FD}	
4	$P_{FD} = 1.000$	
6	$P_{FD} = 0.760 - 0.000025v_F - 0.000046v_R$	Equation 13-9
	$P_{FD} = 0.717 - 0.000039v_F + 0.604(v_U / L_{\mathrm{UP}})$	Equation 13-10
	$P_{FD} = 0.616 - 0.000021v_F + 0.124(v_D / L_{\mathrm{DOWN}})$	Equation 13-11
8	$P_{FD} = 0.436$	

SELECTING EQUATIONS FOR P_{FD} FOR SIX-LANE FREEWAYS

Adjacent Upstream Ramp	Subject Ramp	Adjacent Downstream Ramp	Equation(s) Used
None	Off	None	Equation 13-9
None	Off	On	Equation 13-9
None	Off	Off	Equation 13-11 or 13-9
On	Off	None	Equation 13-10 or 13-9
Off	Off	None	Equation 13-9
On	Off	On	Equation 13-10 or 13-9
On	Off	Off	Equation 13-11, 13-10, or 13-9
Off	Off	On	Equation 13-9
Off	Off	Off	Equation 13-11 or 13-9

Note: [a] 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.
If an adjacent ramp on a six-lane freeway is not a one-lane, right-side off-ramp, use Equation 13-9.

For six-lane freeways, three equations are presented. Equation 13-9 is the base case for isolated ramps or for cases in which the impact of adjacent ramps can be ignored. Equation 13-10 addresses cases in which there is an adjacent upstream on-ramp, while Equation 13-11 addresses cases in which there is an adjacent downstream off-ramp. Adjacent upstream off-ramps and downstream on-ramps have not been found to have a statistically significant impact on diverge operations and may be ignored. All variables in Exhibit 13-7 are as previously defined.

Insufficient information is available to establish an impact of adjacent ramps on eight-lane freeways (four lanes in each direction). This methodology does not include such an impact.

Where an adjacent upstream on-ramp or downstream off-ramp on a six-lane freeway exists, a determination as to whether the ramp is close enough to the subject off-ramp to affect its operation is necessary. As was the case for on-ramps, this is done by finding the equilibrium distance L_{EQ}. This distance is determined when Equation 13-9 yields the same value of P_{FD} as Equation 13-10 (for adjacent upstream on-ramps) or Equation 13-11 (adjacent downstream off-ramps). When the actual distance between ramps is greater than or equal to L_{EQ}, Equation 13-9 is used. When the actual distance between ramps is less than L_{EQ}, Equation 13-10 or Equation 13-11 is used as appropriate.

For adjacent upstream on-ramps, use Equation 13-12 to find the equilibrium distance:

$$L_{EQ} = \frac{v_u}{0.071 + 0.000023 v_F - 0.000076 v_R}$$

Equation 13-12

For adjacent downstream off-ramps, use Equation 13-13:

$$L_{EQ} = \frac{v_D}{1.15 - 0.000032 v_F - 0.000369 v_R}$$

Equation 13-13

where all terms are as previously defined.

A special case exists when both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present. In such cases, two solutions for P_{FD} may arise, depending on which adjacent ramp is considered (both ramps cannot be considered simultaneously). In such cases, the larger value of P_{FD} is used.

When both an adjacent upstream on-ramp and an adjacent downstream off-ramp are present, the larger resulting value of P_{FD} is used.

As was the case for merge areas, the algorithms used to include the impact of an upstream or downstream ramp on a six-lane freeway are only valid for single-lane, right-side adjacent ramps. Where adjacent ramps consist of two-lane junctions or major diverge configurations, or where they are on the left side of the freeway, Equation 13-9 is always applied.

When an adjacent ramp to a diverge area on a six-lane freeway is not a one-lane, right-side ramp, apply Equation 13-9.

Checking the Reasonableness of the Lane Distribution Prediction

The algorithms of Exhibit 13-6 and Exhibit 13-7 were developed through regression analysis of a large database. Unfortunately, regression-based models may yield unreasonable or unexpected results when applied outside the strict limits of the calibration database, and they may have inconsistencies at their boundaries.

Therefore, it is necessary to apply some limits to predicted values of flow in Lanes 1 and 2 (v_{12}). The following limitations apply to all such predictions:

Reasonableness checks on the value of v_{12}.

1. The average flow per lane in the outer lanes of the freeway (lanes other than 1 and 2) should not be higher than 2,700 pc/h/ln.
2. The average flow per lane in outer lanes should not be higher than 1.5 times the average flow in Lanes 1 and 2.

These limits guard against cases in which the predicted value of v_{12} implies an unreasonably high flow rate in outer lanes of the freeway. When either of these limits is violated, an adjusted value of v_{12} must be computed and used in the remainder of the methodology.

Application to Six-Lane Freeways

On a six-lane freeway (three lanes in one direction), there is only one outer lane to consider. The flow rate in this outer lane (Lane 3) is given by Equation 13-14:

Equation 13-14

$$v_3 = v_F - v_{12}$$

where

- v_3 = flow rate in Lane 3 of the freeway (pc/h/ln),
- v_F = flow rate on freeway immediately upstream of the ramp influence area (pc/h), and
- v_{12} = flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h).

Then, if v_3 is greater than 2,700 pc/h, use Equation 13-15:

Equation 13-15

$$v_{12a} = v_F - 2{,}700$$

If v_3 is greater than 1.5 × (v_{12}/2), use Equation 13-16:

Equation 13-16

$$v_{12a} = \left(\frac{v_F}{1.75}\right)$$

where v_{12a} equals the adjusted flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area (pc/h) and all other variables are as previously defined.

In cases where both limitations on outer lane flow rate are violated, the result yielding the highest value of v_{12a} is used. The adjusted value replaces the original value of v_{12} and the analysis continues.

Application to Eight-Lane Freeways

On eight-lane freeways, there are two outer lanes (Lanes 3 and 4). Thus, the limiting values cited previously apply to the average flow rate per lane in these lanes. The average flow in these lanes is computed from Equation 13-17:

Equation 13-17

$$v_{av34} = \frac{v_F - v_{12}}{2}$$

where v_{av34} equals the flow rate in outer lanes (pc/h/ln) and all other variables are as previously defined.

Then, if v_{av34} is greater than 2,700, use Equation 13-18:

Equation 13-18

$$v_{12a} = v_F - 5{,}400$$

If v_{av34} is greater than 1.5 × (v_{12}/2), use Equation 13-19:

Equation 13-19

$$v_{12a} = \left(\frac{v_F}{2.50}\right)$$

where all terms are as previously defined.

In cases where both limitations on outer lane flow rate are violated, the result yielding the highest value of v_{12a} is used. The adjusted value replaces the original value of v_{12} and the analysis continues.

Summary of Step 2

At this point, an appropriate value of v_{12} has been computed and adjusted as necessary.

Step 3: Estimate the Capacity of the Ramp–Freeway Junction and Compare with Demand Flow Rates

There are three major checkpoints for the capacity of a ramp–freeway junction:

Locations for checking the capacity of a ramp–freeway junction.

1. The capacity of the freeway immediately downstream of an on-ramp or immediately upstream of an off-ramp,
2. The capacity of the ramp roadway, and
3. The maximum flow rate entering the ramp influence area.

Freeway capacity immediately downstream of an on-ramp or upstream of an off-ramp is usually the controlling capacity factor.

In most cases, the freeway capacity is the controlling factor. Studies (*1*) have shown that the turbulence in the vicinity of a ramp–freeway junction does not diminish the capacity of the freeway.

Failure of a diverge junction is usually caused by a capacity deficiency at the ramp–street terminal or on the off-ramp roadway.

The capacity of the ramp roadway is rarely a factor at on-ramps, but it can play a major role at off-ramp (diverge) junctions. Failure of a diverge junction is most often caused by a capacity deficiency on the off-ramp roadway or at its ramp–street terminal.

While this methodology establishes a maximum desirable rate of flow entering the ramp influence area, exceeding this value does not cause a failure. Instead, it means that operations may be less desirable than indicated by the methodology. At off-ramps, the total flow rate entering the ramp influence area is merely the estimated value of v_{12}. At on-ramps, however, the on-ramp flow also enters the ramp influence area. Therefore, the total flow entering the ramp influence area at an on-ramp is given by Equation 13-20:

Equation 13-20

$$v_{R12} = v_{12} + v_R$$

where v_{R12} is the total flow rate entering the ramp influence area at an on-ramp (pc/h) and all other variables are as previously defined.

Exhibit 13-8 shows capacity values for ramp–freeway junctions. Exhibit 13-9 shows similar values for high-speed ramps on multilane highways and C-D roadways within freeway interchanges. Exhibit 13-10 shows the capacity of ramp roadways.

Exhibit 13-8
Capacity of Ramp–Freeway Junctions (pc/h)

FFS (mi/h)	Capacity of Upstream/Downstream Freeway Segment[a]				Max. Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Max. Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
	No. of Lanes in One Direction					
	2	3	4	>4		
≥70	4,800	7,200	9,600	2,400/ln	4,600	4,400
65	4,700	7,050	9,400	2,350/ln	4,600	4,400
60	4,600	6,900	9,200	2,300/ln	4,600	4,400
55	4,500	6,750	9,000	2,250/ln	4,600	4,400

Notes: [a] Demand in excess of these capacities results in LOS F.
[b] Demand in excess of these values alone does not result in LOS F; operations may be worse than predicted by this methodology.

Exhibit 13-9
Capacity of High-Speed Ramp Junctions on Multilane Highways and C-D Roadways (pc/h)

FFS (mi/h)	Capacity of Upstream/Downstream Highway or C-D Segment[a]			Max. Desirable Flow Rate (v_{R12}) Entering Merge Influence Area[b]	Max. Desirable Flow Rate (v_{12}) Entering Diverge Influence Area[b]
	No. of Lanes in One Direction				
	2	3	>3		
≥60	4,400	6,600	2,200/ln	4,600	4,400
55	4,200	6,300	2,100/ln	4,600	4,400
50	4,000	6,000	2,000/ln	4,600	4,400
45	3,800	5,700	1,900/ln	4,600	4,400

Notes: [a] Demand in excess of these capacities results in LOS F.
[b] Demand in excess of these values alone does not result in LOS F; operations may be worse than predicted by this methodology.

Exhibit 13-10
Capacity of Ramp Roadways (pc/h)

Ramp FFS S_{FR} (mi/h)	Capacity of Ramp Roadway	
	Single-Lane Ramps	Two-Lane Ramps
>50	2,200	4,400
>40–50	2,100	4,200
>30–40	2,000	4,000
≥20–30	1,900	3,800
<20	1,800	3,600

Note: Capacity of a ramp roadway does not ensure an equal capacity at its freeway or other high-speed junction. Junction capacity must be checked against criteria in Exhibit 13-8 and Exhibit 13-9.

Ramp–Freeway Junction Capacity Checkpoint

As noted previously, it is generally the capacity of the upstream or downstream freeway segment that limits flow through a merge or diverge area, assuming that the number of freeway lanes entering and leaving the ramp junction is the same. In such cases, the critical checkpoint for freeway capacity is

- Immediately downstream of an on-ramp influence area (v_{FO}), or
- Immediately upstream of an off-ramp influence area (v_F).

These are logical checkpoints, since each represents the point at which maximum freeway flow exists.

When a ramp junction or major merge/diverge area involves lane additions or lane drops at the junction, freeway capacity must be checked both immediately upstream and downstream of the ramp influence area.

Failure of any ramp–freeway junction capacity check results in LOS F.

Failure of any ramp–freeway junction capacity check (i.e., demand exceeds capacity: v/c is greater than 1.00) results in LOS F.

Ramp Roadway Capacity Checkpoint

The capacity of the ramp roadway should always be checked against the demand flow rate on the ramp. For on-ramp or merge junctions, this is rarely a problem. Theoretically, cases could exist in which demand exceeds capacity. A failure due to insufficient on-ramp capacity does not, in itself, create problems on the freeway. Rather, it would result in queuing at the streetside terminal of the ramp (or in the case of a freeway-to-freeway ramp, on the entering freeway).

At off-ramps or diverge areas, the most frequent cause of failure is insufficient capacity on the off-ramp—due to either the ramp roadway or a failure of the ramp–street terminal. This methodology checks only for the off-ramp roadway capacity. The capacity of the ramp–street junction must be evaluated by using appropriate methodologies for unsignalized intersections (Chapter 19, 20, or 21) or signalized interchange ramp terminals (Chapter 22).

If the off-ramp demand flow rate v_R exceeds the capacity of the off-ramp, LOS F prevails. If appropriate analysis results in a finding that the ramp–street terminal is operating at a v/c ratio greater than 1.00 on the ramp approach leg, a queuing analysis should be conducted to evaluate (*a*) the extent of the queue that is likely to exist on the ramp roadway and (*b*) whether the queue is close enough to the ramp–freeway junction to affect its operation negatively.

Maximum Desirable Flow Entering the Ramp Influence Area

While a checkpoint for v_{12} (off-ramps) or v_{R12} (on-ramps) is conducted, failure does not result in assignment of LOS F, unless another failure occurs on a ramp roadway or freeway segment. Failing this checkpoint generally means that there will be more turbulence in the ramp junction influence area than predicted by this methodology. Thus, predicted densities are most likely lower than those that will exist, and predicted speeds are most likely higher than those that will actually occur.

Failure of the check for flow entering the ramp influence area (v_{12}, v_{R12}) does not automatically result in LOS F but does indicate the need for additional interpretation of the results.

Step 4: Estimate Density in the Ramp Influence Area and Determine the Prevailing LOS

Once the flow rate in Lanes 1 and 2 immediately upstream of the ramp influence area is determined, the expected density in the ramp influence area can be estimated.

Density in On-Ramp (Merge) Influence Areas

The density in on-ramp influence areas is estimated with Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

Equation 13-21

where D_R is the density in the ramp influence area (pc/mi/ln) and all other variables are as previously defined.

The equation is logical. As more on-ramp vehicles and freeway vehicles in Lanes 1 and 2 enter the ramp influence area, its density is expected to increase. As the length of the acceleration lane increases, there is more physical space in the ramp influence area, and operating speeds of merging vehicles are expected to increase—both tending to reduce densities.

Density in Off-Ramp (Diverge) Influence Areas

The density in off-ramp influence areas is estimated with Equation 13-22:

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

Equation 13-22

where all variables are as previously defined.

This equation also follows logical trends. There is no separate term for v_R because it is included in v_{12} for off-ramps. As the number of vehicles entering the ramp influence area increases, density increases. As the length of the deceleration lane increases, the additional space provided and the resulting higher speeds of merging vehicles both act to reduce density.

Determining LOS

LOS in ramp influence areas is directly related to the estimated density within the area, as given by Equation 13-21 or Equation 13-22. Exhibit 13-2, shown previously, contains the criteria for this determination. Note again that density definitions of LOS apply only to stable flow (i.e., LOS A–E). LOS F exists only when the capacity of the ramp junction is insufficient to accommodate the existing or projected demand flow rate.

If it is determined that a merge or diverge segment is operating (or expected to operate at) LOS F, the analyst should go to Chapter 10, Freeway Facilities, and conduct a facility analysis that will estimate the spatial and time impacts of queuing resulting from the breakdown.

Step 5: Estimate Speeds in the Vicinity of Ramp–Freeway Junctions

While an estimation of average vehicle speeds in and adjacent to ramp influence areas is not necessary, it is often a useful additional performance measure. Two types of speeds may be estimated:

- Average speed of vehicles within the ramp influence area (mi/h), and
- Average speed of vehicles across all lanes (including outer lanes) within the 1,500-ft length of the ramp influence area (mi/h).

Both types of speeds are needed when a freeway facility analysis is conducted (Chapter 10), while the first type of speed provides a useful companion measure to density within the ramp influence area in all cases.

Exhibit 13-11 and Exhibit 13-12 provide equations for estimating the average speed of vehicles (*a*) within the ramp influence area and (*b*) in outer lanes of the freeway adjacent to the 1,500-ft ramp influence area. For four-lane freeways (two lanes in each direction), there are no "outer lanes." For six-lane freeways (three lanes in each direction), there is one outer lane (Lane 3). For eight-lane freeways (four lanes in each direction), there are two outer lanes (Lanes 3 and 4). Exhibit 13-13 provides equations to determine the average speed of all vehicles (ramp plus all freeway vehicles) within the 1,500-ft length of the ramp influence area.

Exhibit 13-11
Estimating Speed at On-Ramp (Merge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS - (FFS - 42)M_S$	
	$M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002\ (L_A S_{FR}/1{,}000)$	
Outer lanes of freeway	$S_O = FFS$	$v_{OA} < 500$ pc/h
	$S_O = FFS - 0.0036(v_{OA} - 500)$	500 pc/h $\le v_{OA} \le$ 2,300 pc/h
	$S_O = FFS - 6.53 - 0.006(v_{OA} - 2{,}300)$	$v_{OA} >$ 2,300 pc/h

Exhibit 13-12
Estimating Speed at Off-Ramp (Diverge) Junctions

Average Speed in	Equation	
Ramp influence area	$S_R = FFS - (FFS - 42)D_S$	
	$D_S = 0.883 + 0.00009 v_R - 0.013 S_{FR}$	
Outer lanes of freeway	$S_O = 1.097 FFS$	$v_{OA} < 1{,}000$ pc/h
	$S_O = 1.097 FFS - 0.0039\left(v_{OA} - 1{,}000\right)$	$v_{OA} \geq 1{,}000$ pc/h

Exhibit 13-13
Estimating Average Speed of All Vehicles at Ramp–Freeway Junctions

Value	Equation
Average flow in outer lanes v_{OA} (pc/h)	$v_{OA} = \dfrac{v_F - v_{12}}{N_O}$
Average speed for on-ramp (merge) junctions (mi/h)	$S = \dfrac{v_{R12} + v_{OA} N_O}{\left(\dfrac{v_{R12}}{S_R}\right) + \left(\dfrac{V_{OA} N_O}{S_O}\right)}$
Average speed for off-ramp (diverge) junctions (mi/h)	$S = \dfrac{v_{12} + v_{OA} N_O}{\left(\dfrac{v_{12}}{S_R}\right) + \left(\dfrac{v_{OA} N_O}{S_O}\right)}$

While many (but not all) of the variables in Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 have been defined previously, all are defined here for convenience:

S_R = average speed of vehicles within the ramp influence area (mi/h); for merge areas, this includes all ramp and freeway vehicles in Lanes 1 and 2; for diverge areas, this includes all vehicles in Lanes 1 and 2;

S_O = average speed of vehicles in outer lanes of the freeway, adjacent to the 1,500-ft ramp influence area (mi/h);

S = average speed of all vehicles in all lanes within the 1,500-ft length covered by the ramp influence area (mi/h);

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

S_{FR} = free-flow speed of the ramp (mi/h);

L_A = length of acceleration lane (ft);

L_D = length of deceleration lane (ft);

v_R = demand flow rate on ramp (pc/h);

v_{12} = demand flow rate in Lanes 1 and 2 of the freeway immediately upstream of the ramp influence area (pc/h);

v_{R12} = total demand flow rate entering the on-ramp influence area, including v_{12} and v_R (pc/h);

v_{OA} = average demand flow per lane in outer lanes adjacent to the ramp influence area (not including flow in Lanes 1 and 2) (pc/h/ln);

v_F = demand flow rate on freeway immediately upstream of the ramp influence area (pc/h);

N_O = number of outer lanes on the freeway (1 for a six-lane freeway; 2 for an eight-lane freeway);

M_S = speed index for on-ramps (merge areas); this is simply an intermediate computation that simplifies the equations; and

D_S = speed index for off-ramps (diverge areas); this is simply an intermediate computation that simplifies the equations.

Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 only apply to stable flow conditions. Consult Chapter 10 for analysis of oversaturated conditions.

The equations in Exhibit 13-11, Exhibit 13-12, and Exhibit 13-13 apply only to cases in which operation is stable (LOS A–E). Analysis of operational details for cases in which LOS F is present relies on deterministic queuing approaches, as presented in Chapter 10, Freeway Facilities.

Flow rates in outer lanes can be higher than the value cited for basic freeway segments. The basic freeway segment values represent averages across all freeway lanes, not flow rates in a single lane or a subset of lanes. The methodology herein allows flows in outer lanes to be as high as 2,700 pc/h/ln. The equations for average speed in outer lanes were based on a database that included average outer lane flows as high as 2,988 pc/h/ln while still maintaining stable flow. Values over 2,700 pc/h/ln, however, are unusual and cannot be expected in the majority of situations.

In addition, the equations of Exhibit 13-11 do not allow a predicted speed over the FFS for merge areas. For diverge areas at low flow rates, however, the average speed in outer lanes may marginally exceed the FFS. As with average lane flow rates, the FFS is stated as an average across all lanes, and speeds in individual lanes can exceed this value. Despite this, the average speed of all vehicles S should be limited to a maximum value equal to the FFS.

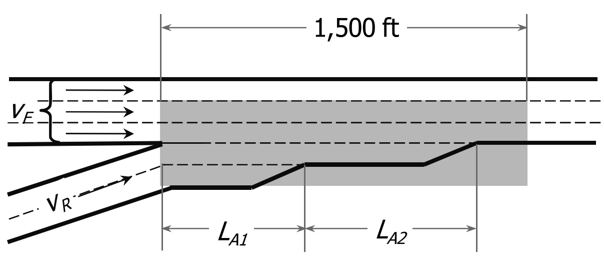
SPECIAL CASES

As noted previously, the computational procedure for ramp–freeway junctions was calibrated for single-lane, right-side ramps. Many other merge and diverge configurations may be encountered, however. In these cases, the general methodology is modified to account for special situations. These modifications are discussed in the sections that follow.

Two-Lane On-Ramps

Exhibit 13-14 illustrates the geometry of a typical two-lane ramp–freeway junction. It is characterized by two separate acceleration lanes, each successively forcing merging maneuvers to the left.

Exhibit 13-14
Typical Geometry of a Two-Lane Ramp–Freeway Junction



Two-lane on-ramps entail two modifications to the basic methodology: the flow remaining in Lanes 1 and 2 immediately upstream of the on-ramp influence area is generally somewhat higher than it is for one-lane on-ramps in similar situations, and densities in the merge influence area are lower than those for similar one-lane on-ramp situations. The lower density is primarily due to the existence of two acceleration lanes and the generally longer distance over which these lanes extend. Thus, two-lane on-ramps handle higher ramp flows more smoothly and at a better LOS than if the same flows were carried on a one-lane ramp–freeway junction.

Two-lane on-ramp–freeway junctions, however, do not enhance the capacity of the junction. The downstream freeway capacity still controls the total output capacity of the merge area, and the maximum desirable number of vehicles entering the ramp influence area is not changed.

There are three computational modifications to the general methodology for two-lane on-ramps.

First, while v_{12} is still estimated as $v_F \times P_{FM}$, the values of P_{FM} are modified as follows:

- For four-lane freeways: P_{FM} = 1.000;
- For six-lane freeways: P_{FM} = 0.555; and
- For eight-lane freeways: P_{FM} = 0.209.

Second, in all equations using the length of the acceleration lane L_A, this value is replaced by the effective length of both acceleration lanes L_{Aeff} from Equation 13-23:

$$L_{Aeff} = 2L_{A1} + L_{A2}$$

Equation 13-23

A two-lane ramp is always considered to be isolated (i.e., no adjacent ramp conditions affect the computation).

Component lengths are as illustrated in Exhibit 13-14.

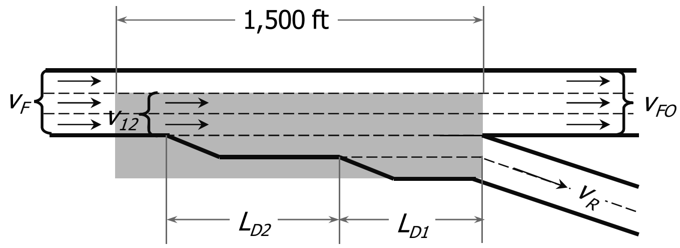
Two-Lane Off-Ramps

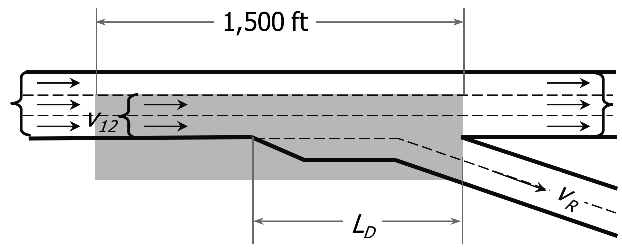
Two common types of diverge geometries are in use with two-lane off-ramps, as shown in Exhibit 13-15. In the first, two successive deceleration lanes are introduced. In the second, a single deceleration lane is used. The left-hand ramp lane splits from Lane 1 of the freeway at the gore area, without a deceleration lane.

As is the case for two-lane on-ramps, there are three computational step modifications. While v_{12} is still computed as $v_R + (v_F - v_R) \times P_{FD}$, the values of P_{FD} are modified as follows:

- For four-lane freeways: P_{FD} = 1.000;
- For six-lane freeways: P_{FD} = 0.450; and
- For eight-lane freeways: P_{FD} = 0.260.

Exhibit 13-15
Common Geometries for Two-Lane Off-Ramp–Freeway Junctions





Where a single deceleration lane is used, there is no modification to the length of the deceleration lane L_D; where two deceleration lanes exist, the length is replaced by the effective length L_{Deff} in all equations, obtained from Equation 13-24:

Equation 13-24

$$L_{Deff} = 2L_{D1} + L_{D2}$$

A two-lane ramp is always considered to be isolated (i.e., no adjacent ramp conditions affect the computation).

Component lengths are as illustrated in Exhibit 13-15.

The capacity of a two-lane off-ramp is essentially equal to that of a similar one-lane off-ramp.

The capacity of a two-lane off-ramp freeway junction is essentially equal to that of a similar one-lane off-ramp; that is, the total flow capacity through the diverge is unchanged. It is limited by the upstream freeway, the downstream freeway, or the off-ramp capacity. While the capacity is not affected by the presence of two-lane junctions, the lane distribution of vehicles is more flexible than in a similar one-lane case. The two-lane junction may also be able to accommodate a higher off-ramp flow than can a single-lane off-ramp.

Left-Hand On- and Off-Ramps

While they are not normally recommended, left-hand ramp–freeway junctions do exist on some freeways, and they occur frequently on C-D roadways. The left-hand ramp influence area covers the same 1,500-ft length as that of right-hand ramps—upstream of off-ramps; downstream of on-ramps.

For right-hand ramps, the ramp influence area involves Lanes 1 and 2 of the freeway. For left-hand ramps, the ramp influence area involves the two leftmost lanes of the freeway. For four-lane freeways (two lanes in each direction), this does not involve any changes, since only Lanes 1 and 2 exist. For six-lane freeways (three lanes in each direction), the flow in Lanes 2 and 3 (v_{23}) is involved. For eight-lane freeways (four lanes in each direction), the flow in Lanes 3 and 4 (v_{34}) is involved.

While there is no direct methodology for the analysis of left-hand ramps, some rational modifications can be applied to the right-hand ramp methodology to produce reasonable results (*3*).

It is suggested that analysts compute v_{12} as if the ramp were on the right. An estimate of the appropriate flow rate in the two leftmost lanes is then obtained by multiplying the result by the adjustment factors shown in Exhibit 13-16.

Exhibit 13-16
Adjustment Factors for Left-Hand Ramp–Freeway Junctions

Freeway Size	Adjustment Factor for Left-Hand Ramps: On-Ramps	Off-Ramps
Four-lane	1.00	1.00
Six-lane	1.12	1.05
Eight-lane	1.20	1.10

The remaining computations for density and speed continue by using the value of v_{23} (six-lane freeways) or v_{34} (eight-lane freeways), as appropriate. All capacity values remain unchanged.

Ramp–Freeway Junctions on 10-Lane Freeways (Five Lanes in Each Direction)

Freeway segments with five continuous lanes in a single direction are becoming more common in North America. A procedure is therefore needed to analyze a single-lane, right-hand on- or off-ramp on such a segment.

The approach taken is relatively simple: estimate the flow in Lane 5 of such a segment and deduct it from the approaching freeway flow v_F. With the Lane 5 flow deducted, the segment can now be treated as if it were an eight-lane freeway (*4*). Exhibit 13-17 shows the recommended values for flow rate in Lane 5 of these segments.

Exhibit 13-17
Expected Flow in Lane 5 of a 10-Lane Freeway Immediately Upstream of a Ramp–Freeway Junction

On-Ramps: Approaching Freeway Flow v_F (pc/h)	On-Ramps: Approaching Lane 5 Flow v_5 (pc/h)	Off-Ramps: Approaching Freeway Flow v_F (pc/h)	Off-Ramps: Approaching Lane 5 Flow v_5 (pc/h)
≥8,500	2,500	≥7,000	0.200 v_F
7,500–8,499	0.285 v_F	5,500–6,999	0.150 v_F
6,500–7,499	0.270 v_F	4,000–5,499	0.100 v_F
5,500–6,499	0.240 v_F	<4,000	0
<5,500	0.220 v_F		

Once the expected flow in Lane 5 is determined, the effective total freeway flow rate in the remaining four lanes is computed from Equation 13-25:

Equation 13-25

$$v_{F4eff} = v_F - v_5$$

where

v_{F4eff} = effective approaching freeway flow in four lanes (pc/h),

v_F = total approaching freeway flow in five lanes (pc/h), and

v_5 = estimated approaching freeway flow in Lane 5 (pc/h).

The remainder of the analysis uses the adjusted approaching freeway flow rate and treats the geometry as if it were a single-lane, right-hand ramp junction on an eight-lane freeway (four lanes in each direction).

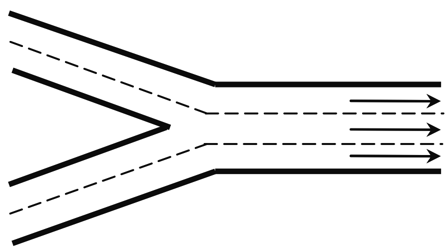
There is no calibrated procedure for adapting the methodology of this chapter to freeways with more than five lanes in one direction. The approach of Equation 13-25 is, however, conceptually adaptable to such situations. A local calibration of the amount of traffic using Lanes 5+ would be needed. The remaining flow could then be modeled as if it were taking place on a four-lane (one direction) segment.

Major Merge Areas

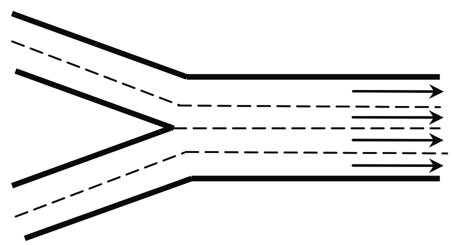
A major merge area is one in which two primary roadways, each having multiple lanes, merge to form a single freeway segment. Such junctions occur when two freeways join to form a single freeway or when a major multilane high-speed ramp joins with a freeway. Major merges are different from one- and two-lane on-ramps in that each of the merging roadways is generally at or near freeway design standards and no clear ramp or acceleration lane is involved in the merge.

Such merge areas come in a variety of geometries, all of which fall into one of two categories. In one geometry, the number of lanes leaving the merge area is one less than the total number of lanes entering it. In the other, the number of lanes leaving the merge area is the same as that entering it. These geometries are illustrated in Exhibit 13-18.

Exhibit 13-18
Major Merge Areas Illustrated



(a) Major Merge with One Lane Dropped



(b) Major Merge with No Lane Dropped

LOS cannot be determined for major merge areas.

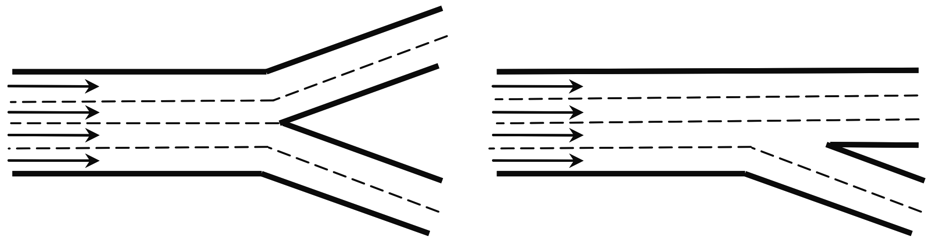
There are no effective models of performance for a major merge area. Therefore, analysis is limited to checking capacities on the approaching legs and the downstream freeway segment. A merge failure would be indicated by a v/c ratio in excess of 1.00. LOS cannot be determined for major merge areas. Problems in major merge areas usually result from insufficient capacity of the downstream freeway segment.

Major Diverge Areas

The two common geometries for major diverge areas are illustrated in Exhibit 13-19. In the first case, the number of lanes leaving the diverge area is the same as the number entering it. In the second, the number of lanes leaving the diverge area is one more than the number entering it.

The principal analysis of a major diverge area involves checking the capacity of entering and departing roadways, all of which are generally built to mainline standards. A failure results when any of the demand flow rates exceeds the capacity of the segment.

Exhibit 13-19
Major Diverge Areas Illustrated



(a) Major Diverge Area with No Lane Addition (b) Major Diverge Area with Lane Addition

For major diverge areas, a model exists for computing the average density across all approaching freeway lanes within 1,500 ft of the diverge, as given in Equation 13-26:

Equation 13-26

$$D_{MD} = 0.0175\left(\frac{v_F}{N}\right)$$

where

D_{MD} = density in the major diverge influence area (which includes all approaching freeway lanes) (pc/mi/ln),

v_F = demand flow rate immediately upstream of the major diverge influence area (pc/h), and

N = number of lanes approaching the major diverge.

The result can be compared with the criteria of Exhibit 13-2 to determine a LOS for the major diverge influence area. Note that the density and LOS estimates are only valid for stable cases (i.e., not in cases in which LOS F exists because of a capacity deficiency on the approaching or departing legs of the diverge).

Effect of Ramp Control at On-Ramps

For the purposes of this methodology, procedures are not modified in any way to account for the local effect of ramp control—except for the limitation that the ramp meter may have on the ramp demand flow rate. Research (*5*) has found that the breakdown of a merge area may be a probabilistic event based on the platoon characteristics of the arriving ramp vehicles. Ramp meters facilitate uniform gaps between entering ramp vehicles and may reduce the probability of a breakdown on the associated freeway mainline.

OVERLAPPING RAMP INFLUENCE AREAS

Whenever a series of ramps on a freeway is analyzed, the 1,500-ft ramp influence areas could overlap. In such cases, the operation in the overlapping region is determined by the ramp influence area having the highest density.

3. APPLICATIONS

The methodology of this chapter is most often used to estimate the capacity and LOS of ramp–freeway junctions. The steps are most easily applied in the operational analysis mode (i.e., all traffic and roadway conditions are specified), and the capacity (and *v*/*c* ratio) and expected LOS are found. Other types of analysis, however, are possible.

DEFAULT VALUES

A comprehensive presentation of potential default values for uninterrupted-flow facilities is provided elsewhere (*6*). Chapter 10, Freeway Facilities, provides a summary of the default values for freeways. These defaults cover the key characteristics of peak hour factor (PHF) and percent heavy vehicles (%HV) on freeways. Recommendations are based on geographical region, population, and time of day. All general freeway default values may be applied to the analysis of ramp–freeway junctions in the absence of field data or projections of conditions.

Because of the number of variables involved in the analysis of ramps, which have been discussed previously, it is difficult to base an analysis on too many default values. Clearly, all demand flow rates must be specified, even if they are projections.

Ramp geometric characteristics cover a variety of conditions; default values should be avoided if possible.

Similarly, geometric characteristics of ramps cover a wide variety of conditions. If absolutely necessary, the following additional default values may be applied to a ramp junction analysis:

- Length of acceleration lane L_A = 800 ft,
- Length of deceleration lane L_D = 400 ft,
- FFS of ramp S_{FR} = 35 mi/h, and
- Driver population factor f_P = 1.00.

Obviously, as the number of default values used in any analysis increases, the accuracy of the result becomes more approximate, and the result may be significantly different from the actual outcome (depending on local conditions). If locally calibrated default values are available, they may be substituted for the values above.

ESTABLISH ANALYSIS BOUNDARIES

No ramp–freeway junction is completely isolated. However, for the purposes of this methodology, many may operate as if they were. In the analysis of ramp–freeway junctions, it is important to establish the segment of freeway over which ramp junctions are to be analyzed. Once this is done, each ramp may be analyzed in conjunction with the possible impacts of upstream and downstream adjacent ramps according to the methodology.

Analysis boundaries may also include different demand scenarios related to the time of the day or to different development scenarios that produce different demand flow rates.

Any application of the methodology presented in this chapter can be made easier by carefully defining the spatial and time boundaries of the analysis.

TYPES OF ANALYSIS

The methodology of this chapter can be used in three types of analysis: operational analysis, design analysis, and planning and preliminary design analysis.

Operational Analysis

Operational analysis determines density, LOS, and speed within the ramp influence area for a specified set of conditions.

The methodology is most easily applied in the operational analysis mode. In operational analysis, all traffic and geometric characteristics of the analysis segment must be specified, including

- Analysis hour demand volumes for the subject ramp, adjacent ramps, and freeway (veh/h);
- Heavy vehicle percentages for all component demand volumes (ramps, adjacent ramps, freeway);
- PHF for all component demand volumes (ramp, adjacent ramps, freeway);
- Freeway terrain (level, rolling, mountainous, specific grade);
- FFS of the freeway and ramp (mi/h);
- Ramp geometrics: number of lanes, terrain, length of acceleration lane(s) or deceleration lane(s); and
- Distance to upstream and downstream adjacent ramps (ft).

The outputs of an operational analysis will be estimates of density, LOS, and speed for the ramp influence area. The capacity of the ramp–freeway junction will also be established.

The steps of the methodology, described in the Methodology section, are to be followed directly without modification.

Design Analysis

Design analysis seeks to determine the geometric characteristics of the ramp that are needed to deliver a target LOS.

In design analysis, a target LOS is set and all relevant demand volumes are specified. The analysis seeks to determine the geometric characteristics of the ramp that are needed to deliver the target LOS. These characteristics include

- FFS of the ramp S_{FR} (mi/h),
- Length of acceleration L_A or deceleration lane L_D (ft), and
- Number of lanes on the ramp.

In some cases, variables such as the type of junction (e.g., major merge, two-lane) may also be under consideration.

There is no convenient way to compute directly the optimal value of any one variable without specifying all of the others. Even then, the computational methodology does not easily create the desired result.

Therefore, most design analysis becomes a trial-and-error application of the operational analysis procedure. Individual characteristics can be incrementally changed, as can groups of characteristics, to find scenarios that produce the desired LOS.

In many cases, some of the variables may be fixed by site-specific conditions. These can be set at their limiting values before attempting to optimize the others.

It is possible to program a spreadsheet to complete such an analysis, providing scenario results by simply changing some of the input variables under consideration. HCM-implementing software can also be used to simplify the computational process.

Planning and Preliminary Engineering Analysis

Planning and preliminary engineering analysis also seeks to determine the geometric characteristics of the ramp that are needed to deliver a target LOS, but it relies on more general input data.

The desired outputs of planning and preliminary engineering analysis are virtually the same as those for design analysis. The primary difference is that planning and preliminary engineering analysis occurs very early in the process of project consideration.

The first criterion that categorizes such applications is the need to use more general estimates of input data. Many of the default values specified for freeway facilities in Chapter 10 would be applied; alternatively, local default values can be substituted. Demand volumes might be specified only as expected values of annual average daily traffic (AADT) for a target year. Directional design-hour volumes are based on AADTs; default (local or global) values are used for the *K*-factor (the proportion of AADT occurring in the peak hour) and the *D*-factor (the proportion of peak hour traffic traveling in the peak direction). Guidance on these values is given in Chapter 3, Modal Characteristics.

On the basis of these default and estimated values, the analysis is conducted in the same manner as a design analysis.

Service Volumes and Service Flow Rates

The method can be applied to determine service volumes for LOS A–E for a specified set of conditions.

Service volume is the maximum hourly volume that can be accommodated without exceeding the limits of the various levels of service during the worst 15 min of the analysis hour. Service volumes can be found for LOS A–E. LOS F, which represents unstable flow, does not have a service volume.

Service flow rates are the maximum rates of flow (within a 15-min period) that can be accommodated without exceeding the limits of the various levels of service. As is the case for service volumes, service flow rates can be found for LOS A–E, but none is defined for LOS F. The relationship between a service volume and a service flow rate is as follows:

Equation 13-27

$$SV_i = SF_i \times PHF$$

where

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

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

PHF = peak hour factor.

For ramp–freeway junctions, service flow rate or service volume could be defined in several ways. It might be argued that since ramp–freeway junction capacities are usually limited by the upstream or downstream freeway segment, service flow rates and service volumes should be based on basic freeway criteria applied to the upstream or downstream freeway segments. This, however, would ignore the levels of service defined for the ramp influence area, which are the only unique service descriptors for ramps.

Levels of service for ramp–freeway junctions are defined in Exhibit 13-2 and relate to the density within the ramp influence area. The methodology estimates this density by using a series of algorithms affected by demand flows on the freeway, ramp, and adjacent ramps; ramp geometrics; and distances to adjacent ramps. The methodology uses demand volumes in vehicles per hour converted to demand flow rates in passenger cars per hour. Therefore, service flow rates and service volumes would originally be estimated in terms of flow rates in passenger cars per hour. They would then be converted back to demand volumes in vehicles per hour.

Because the balance of ramp and freeway demands has a significant impact on densities, there are a number of ways in which service flow rates and volumes can be considered:

- The limiting total upstream demand volume that produces a given LOS within the ramp influence area. The split between arriving freeway volume and ramp volume would have to be specified.
- The limiting volume entering the ramp influence area that produces a given LOS within the ramp influence area. Since this relies on the approaching freeway volume, the split between freeway and ramp demand would still have to be specified.
- The limiting ramp volume that produces a given LOS within the ramp influence area, based on a fixed upstream freeway demand.

Any of these are viable concepts for establishing a ramp service flow rate or service volume.

A number of factors influence the service volume or flow rate result; each situation must be individually considered.

In addition to different ways of interpreting a service volume or service flow rate, a large number of characteristics will influence the result, including the PHF, %HV, length of acceleration or deceleration lane(s), ramp FFS, and any relevant data for adjacent ramps. It is, therefore, virtually impossible to define a representative "typical" case with broadly applicable results. Each case must be individually considered.

The Example Problems section includes an example of how ramp junction service flow rates and service volumes can be computed.

USE OF ALTERNATIVE TOOLS

General guidance for the use of alternative traffic analysis tools for capacity and LOS analysis is provided in Chapter 6, HCM and Alternative Analysis Tools. This section contains specific guidance for applying alternative tools to the analysis of ramps and ramp junctions. Additional information on this topic may be found in the Volume 4 Technical Reference Library.

The HCM methodology for analyzing merge and diverge segments estimates the density of the ramp influence area (which includes the two rightmost lanes of the freeway and the acceleration or deceleration lane) and provides the respective LOS. As an intermediate step, the methodology estimates the capacity at various points through the section, and if the capacity is exceeded, the LOS is determined to be F without further calculation of density. The methodology is primarily based on the estimation of the demand into the influence area v_{12}.

Strengths of the HCM Procedure

This chapter's procedures were developed on the basis of extensive research supported by a significant quantity of field data. They have evolved over a number of years and represent a body of expert consensus. Most simulation packages will not include the level of detail present in this methodology concerning the ramp itself and its adjacent upstream and downstream ramps.

The HCM procedure's strengths are as follows:

- The methodology provides capacity estimates. Simulators do not provide capacity estimates directly; they can be obtained by devising a data collection scheme in the simulator. Furthermore, the user can modify those simulated capacities by modifying specific input values, such as the minimum acceptable headway.
- The methodology explicitly considers the impacts of the presence of and demands on the upstream and downstream ramps.
- It produces a single deterministic estimate of density, which is important for some purposes, such as development impact review.

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

A list of the HCM's limitations for freeway merge and diverge segments is provided in Exhibit 13-20.

Exhibit 13-20
Limitations of the HCM Ramps and Ramp Junctions Procedure

Limitation	Potential for Improved Treatment by Alternative Tools
Managed lanes, such as HOV lanes, as ramp entrance lanes	Modeled explicitly by simulation
Ramp metering	Modeled explicitly by simulation
Oversaturated conditions (*Refer to Chapter 10 for further discussion*)	Modeled explicitly by simulation
Posted speed limit and extent of police enforcement	Can be approximated by using assumptions related to the desired speed along a given segment
Presence of intelligent transportation system features	Several features modeled explicitly by simulation; others may be approximated by using assumptions (for example, by modifying origin–destination demands by time interval)
Freeway operational analysis beyond the 1,500-ft area of influence	Modeled explicitly by simulation
Capacity-enhancing effects of ramp metering	Can be approximated by using assumptions related to car-following, lane-changing, and gap-acceptance behavior

Ramp junctions can also be analyzed with a variety of stochastic and deterministic simulation packages that address freeways. These packages can be useful in analyzing the extent of congestion when there are failures either within or downstream of the simulated facility range.

Additional Features and Performance Measures Available From Alternative Tools

This chapter provides a methodology for estimating the capacity, speed, and density in the area of influence of on- and off-ramps, given traffic demands and segment characteristics. Alternative tools offer additional performance measures including delay, stops, queue lengths, fuel consumption, pollution, and operating costs.

As with most other HCM procedural chapters, simulation outputs, especially graphics-based presentations, can provide details on point problems that might otherwise go unnoticed with a macroscopic analysis that yields only segment-level measures. The effect of downstream conditions on lane utilization and backup beyond the segment boundary is a good example of a situation that can benefit from the increased insight offered by a microscopic model.

Development of HCM-Compatible Performance Measures Using Alternative Tools

The subject of performance-measure comparisons was discussed in more detail in Chapter 7, Interpreting HCM and Alternative Tool Results. This section deals with topics that apply specifically to ramps and ramp junctions.

When alternative tools are used, the analyst must be careful to note the definitions of simulation outputs. This chapter's measure of effectiveness for ramps and ramp junctions is the density of the ramp influence area. However, most simulators do not provide density estimates separately for the two rightmost lanes within a link. This is a potentially significant obstacle in obtaining the service measures for ramp junctions from a simulator (unless the freeway has only two lanes per direction). Furthermore, in a simulator, there are lane changes along the entire segment. Therefore, it is not clear how a simulator should address the partial presence of vehicles in the link to ensure compatibility with the HCM. Also, as is generally the case for basic freeway segments, increased speed variability in driver behavior (which simulators usually include) results in lower average space mean speed and higher density.

Most simulation packages do not provide separate density estimates for the two right-hand lanes within a link, which is a potentially significant obstacle in obtaining service measures.

In obtaining density from alternative models, it is important to consider the following:

- The ability of the simulator to provide density for the two rightmost lanes of the freeway;
- The vehicles included in the density estimation and how partial presence of vehicles on the link is considered;
- The manner in which the acceleration and deceleration lanes are considered in the density estimation;
- The units used by the simulator to measure density [most use vehicles rather than passenger cars; converting vehicles to passenger cars by using

the HCM's passenger-car equivalence (PCE) values is typically not appropriate, given that simulator assumptions with regard to heavy vehicle performance vary widely];

- The units used in the reporting of density (i.e., whether density is reported per lane mile);
- The homogeneity of the analysis segment in the simulator, as the HCM assumes conditions to be homogeneous (unless it is a specific upgrade or downgrade segment, in which case the segment length is used to estimate the PCE values); and
- The treatment of driver variability by the simulator, as increased driver variability in the simulator will generally increase the average density.

With regard to capacity, the HCM provides capacity estimates in units of passenger cars per hour per lane as a function of FFS for the locations approaching and departing the merge junction. In comparing the HCM estimates with capacity estimates from a simulator, the following should be considered:

- The manner in which a simulator provides the number of vehicles exiting a segment. In some cases it may be necessary to provide virtual detectors at specific points on the simulated segment so that the maximum throughput can be obtained.
- The simulator provides the maximum throughput at a particular location in units of vehicles, rather than passenger cars. Converting these units to passenger cars by using the HCM's PCE values is typically not appropriate, given that simulator assumptions with regard to heavy vehicle performance vary widely.
- A simulator will likely include inputs such as the "minimum separation of vehicles," which greatly affects the maximum throughput.

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

Ramp junction density does not change with FFS in the HCM method, but density is a function of FFS in most simulation packages.

In the HCM, the density at a ramp junction does not change with FFS, although density drops as a function of FFS on basic freeway segments. In simulators, the density typically changes as a function of FFS (or the desired speed). Therefore, calibration of a site using a specific FFS does not necessarily ensure that the site will be calibrated for a different FFS. Capacity, on the other hand, increases in the HCM with increasing FFS, which is typically the case with simulators.

The HCM method is based on the estimated demand approaching the ramp influence area. This demand is estimated as a function of the presence of and demands on the upstream and downstream ramps. Traffic simulators do not typically allow the user to input the specific percentages of traffic on each lane at the beginning of a link. Their internal rules relative to the lane chosen by a vehicle in a given link vary widely and can be modified by changing various default values within the simulator. In some simulators, virtual vehicles are "aware" of their ultimate destination; in others, the exit choice is made on a link-by-link basis. Therefore, in comparing HCM results with those of a simulator, the analyst should, as an intermediate check, compare the flow approaching the two rightmost lanes of the junction.

Adjustment of Simulation Parameters to the HCM Results

The most important elements to be adjusted in analyzing a ramp junction are as follows:

- The flow approaching the two rightmost lanes (this is an intermediate step but would ensure that the influence of upstream and downstream ramps is considered in a manner compatible with the HCM), and
- The capacity of the junction at the critical locations indicated in the HCM (i.e., downstream of the junction and approaching the influence area).

Step-by-Step Recommendations for Applying Alternative Tools

The following steps are recommended when an alternative tool is applied to the analysis of ramps and ramp junctions:

1. Determine whether the chosen tool can provide density for the two rightmost lanes of the freeway and what approach is used to obtain it (including the treatment of the partial presence of vehicles on the link).
2. Determine the FFS of the study site, either from field data or by estimating it according to the Chapter 11 method for basic freeway segments.
3. Enter all available input characteristics (both geometric and traffic characteristics) into the simulator. The length of the segment or link to be simulated should be 1,500 ft, to correspond to the HCM-defined area of influence. Install virtual detectors within the area of influence and at the downstream end of the study segment to obtain density, speeds, and flows.
4. Load the study network above capacity to obtain the maximum throughput, and compare the result with the HCM estimate. Calibrate the simulator by modifying parameters related to the minimum time headway so that the simulated capacity matches the HCM estimate. Estimate the required number of simulation runs that will need to be conducted to produce a statistically valid comparison.
5. Compare the flow approaching the two rightmost lanes with the HCM's estimate. Adjust the simulation parameters related to driver awareness of upcoming turns to match the HCM-predicted v_{12} value.

Example Problems Illustrating Alternative Tool Applications

Chapter 28, Freeway Merges and Diverges: Supplemental, includes two example problems that examine situations beyond the scope of this chapter's methodology by using a typical microsimulation-based tool. Both problems are based on this chapter's Example Problem 3, which analyzes an eight-lane freeway segment with an entrance and an exit ramp. The first problem evaluates the effects of the addition of ramp metering, while the second evaluates the impacts of converting the leftmost lane of the mainline into an HOV lane.

4. EXAMPLE PROBLEMS

Exhibit 13-21
List of Example Problems

Example Problem	Title	Type of Analysis
1	Isolated One-Lane, Right-Hand On-Ramp to a Four-Lane Freeway	Operational analysis
2	Two Adjacent Single-Lane, Right-Hand Off-Ramps on a Six-Lane Freeway	Operational analysis
3	One-Lane On-Ramp Followed by a One-Lane Off-Ramp on an Eight-Lane Freeway	Operational analysis
4	Single-Lane, Left-Hand On-Ramp on a Six-Lane Freeway	Special case
5	Service Flow Rates and Service Volumes for an Isolated On-Ramp on a Six-Lane Freeway	Service flow rates and service volumes

EXAMPLE PROBLEM 1: ISOLATED ONE-LANE, RIGHT-HAND ON-RAMP TO A FOUR-LANE FREEWAY

The Facts

The following data are available to describe the traffic and geometric characteristics of this location:

- Isolated location (no adjacent ramps to consider)
- One-lane ramp roadway and junction
- Four-lane freeway (two lanes in each direction)
- Upstream freeway demand volume = 2,500 veh/h
- Ramp demand volume = 550 veh/h
- 10% trucks, 0% RVs on the freeway
- 5% trucks, 0% RVs on the ramp
- Acceleration lane = 740 ft
- FFS, freeway = 60 mi/h
- FFS, ramp = 45 mi/h
- Level terrain for freeway and ramp
- Peak hour factor = 0.90
- Drivers are regular commuters

Comments

All input parameters are known, so no default values are needed or used. Adjustment factors for heavy vehicles and driver population are found in Chapter 11, Basic Freeway Segments.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

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

Demand volumes are given for the freeway and the ramp. The PHF is specified. The driver population factor for commuters is 1.00 (Chapter 11), while the heavy vehicle adjustment factor is computed as follows:

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

Truck and RV presence is given. The value of E_T for level terrain is 1.5 (Chapter 11). On the basis of these values, the freeway and ramp demand volumes are converted as follows:

For the freeway:

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

$$v_F = \frac{2{,}500}{0.90 \times 0.952 \times 1.00} = 2{,}918 \text{ pc/h}$$

For the ramp:

$$f_{HV} = \frac{1}{1 + 0.05\,(1.5 - 1)} = 0.976$$

$$v_F = \frac{550}{0.90 \times 0.976 \times 1.00} = 626 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Ramp Influence Area with Equation 13-2 and Exhibit 13-6

$$v_{12} = v_F \times P_{FM}$$

The freeway flow rate was computed in Step 1. The value of P_{FM} is found in Exhibit 13-6. For a four-lane freeway, the value is 1.00. Then

$$v_{12} = 2{,}918 \times 1.00 = 2{,}918 \text{ pc/h}$$

Because there are no outer lanes on a four-lane freeway, there is no need to check this result for reasonableness.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

The critical capacity checkpoint for a single-lane on-ramp is the downstream freeway segment:

$$v_{FO} = v_F + v_R = 2{,}918 + 626 = 3{,}544 \text{ pc/h}$$

The capacity of a four-lane freeway (two lanes in one direction) with an FFS of 60 mi/h is given in Exhibit 13-8. The capacity is 4,600 pc/h, which is more than the demand flow of 3,544 pc/h. The capacity of a one-lane ramp with an FFS of 45 mi/h is given in Exhibit 13-10 as 2,100 pc/h, which is well in excess of the ramp demand flow of 626 pc/h. The maximum desirable flow rate entering the ramp influence area is also 4,600 pc/h, again more than 3,544. Thus, the operation of the segment is expected to be stable. LOS F does not exist.

Step 4: Compute Density and Find LOS by Using Equation 13-21 and Exhibit 13-2

The estimated density in the ramp–freeway junction is estimated by using Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

$$D_R = 5.475 + (0.00734 \times 626) + (0.0078 \times 2{,}918) - (0.00627 \times 740) = 28.2 \text{ pc/mi/ln}$$

From Exhibit 13-2, this is LOS D, but the result is close to the LOS C boundary.

Step 5: Compute Merge Area Speed as Supplemental Information by Using Exhibit 13-11

Since there are no outer lanes present on a four-lane freeway, only the speed within the ramp influence area should be computed:

$$S_R = FFS - (FFS - 42) M_S$$

$$M_S = 0.321 + 0.0039 e^{(v_{R12}/1{,}000)} - 0.002\left(L_A S_{FR}/1{,}000\right)$$

$$M_S = 0.321 + 0.0039 e^{(3{,}544/1{,}000)} - 0.002(740 \times 45/1{,}000) = 0.389$$

$$S_R = 60 - (60 - 42) \times 0.389 = 53.0 \text{ mi/h}$$

Discussion

The results indicate that the merge area operates in a stable fashion, with some deterioration in density and speed due to merging operations.

EXAMPLE PROBLEM 2: TWO ADJACENT SINGLE-LANE, RIGHT-HAND OFF-RAMPS ON A SIX-LANE FREEWAY

The Facts

The following information concerning demand volumes and geometries is available for this problem:

- Two consecutive one-lane, right-hand off-ramps
- Six-lane freeway with FFS = 60 mi/h
- Rolling terrain for freeway and both ramps
- 5% trucks on freeway and both ramps; 0% RVs
- First ramp FFS = 40 mi/h
- Second ramp FFS = 25 mi/h
- Drivers are regular commuters
- Freeway demand volume = 4,500 veh/h (immediately upstream of the first off-ramp)
- First ramp demand volume = 300 veh/h
- Second ramp demand volume = 500 veh/h
- Distance between ramps = 750 ft
- First ramp deceleration lane length = 500 ft

- Second ramp deceleration lane length = 300 ft
- Peak hour factor = 0.95

Comments

The solution will use adjustment factors for heavy vehicle presence and driver population selected from Chapter 11, Basic Freeway Segments. All input parameters are specified, so no default values are needed or used.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

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

In this case, three demand volumes must be converted: the freeway volume immediately upstream of the first ramp and the two ramp demand volumes. Since all demands include 5% trucks and no RVs, only a single heavy vehicle adjustment factor will be needed. From Chapter 11, the appropriate value of E_T for rolling terrain is 2.5. For drivers who are regular commuters, the appropriate value of f_p is 1.00.

Then

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

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

and

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

$$v_F = \frac{4{,}500}{0.95 \times 0.930 \times 1.00} = 5{,}093 \text{ pc/h}$$

$$v_{R1} = \frac{300}{0.95 \times 0.930 \times 1.00} = 340 \text{ pc/h}$$

$$v_{R2} = \frac{500}{0.95 \times 0.930 \times 1.00} = 566 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Two Ramp Influence Areas by Using Equation 13-13 and Exhibit 13-7

Because there are two consecutive off-ramps under consideration, the first will have to consider the impact of the second on its operations, and the second will have to consider the impact of the first.

First Off-Ramp

From Exhibit 13-7, flow in Lanes 1 and 2 of the freeway is estimated by using Equation 13-11 or Equation 13-9, depending on whether the impact of the downstream off-ramp is significant. This is determined by computing the equivalence distance by using Equation 13-13:

$$L_{EQ} = \frac{v_D}{1.15 + 0.000032 v_F - 0.000369 v_{R1}}$$

$$L_{EQ} = \frac{566}{1.15 + (0.000032 \times 5{,}093) - (0.000369 \times 340)} = 657 \text{ ft}$$

Since the actual distance between ramps, 750 ft, is greater than the equivalence distance of 657 ft, the ramp may be treated as if it were isolated, with Equation 13-9:

$$P_{FD} = 0.760 - 0.000025 v_F - 0.000046 v_{R1}$$

$$P_{FD} = 0.760 - (0.000025 \times 5093) - (0.000046 \times 340) = 0.617$$

Then

$$v_{12} = v_R + (v_F - v_R) P_{FD}$$

$$v_{12} = 340 + (5{,}093 - 340) \times 0.617 = 3{,}273 \text{ pc/h}$$

Because a six-lane freeway includes one outer lane (Lane 3), the reasonableness of the predicted lane distribution of arriving freeway vehicles should be checked. The flow rate in Lane 3 is 5,093 – 3,273 = 1,820 pc/h. The average flow per lane in Lanes 1 and 2 is 3,273/2 = 1,637 pc/h (rounded to the nearest pc). Then:

Is v_3 > 2,700 pc/h/ln?	**No**
Is v_3 > 1.5 × (1,637) = 2,456 pc/h/ln?	**No**

Since both checks for reasonable lane distribution are passed, the computed value of v_{12} for the first off-ramp is accepted as 3,273 pc/h.

Second Off-Ramp

From Exhibit 13-7, the second off-ramp should be analyzed by using Equation 13-9, which is for an isolated off-ramp. Adjacent upstream off-ramps do not affect the lane distribution of arriving vehicles at a downstream off-ramp.

The freeway flow approaching Ramp 2, however, includes the freeway flow approaching Ramp 1, less the flow rate of vehicles exiting the freeway at Ramp 1. Therefore, the freeway flow rate approaching Ramp 2 is as follows:

$$v_{F2} = 5{,}093 - 340 = 4{,}753 \text{ pc/h}$$

Then

$$P_{FD} = 0.760 - (0.000025 \times 4753) - (0.000046 \times 566) = 0.615$$

$$v_{12} = 566 + (4{,}753 - 566) \times 0.615 = 3{,}141 \text{ pc/h}$$

Again, because there is an outer lane on a six-lane freeway, the reasonableness of this estimate must be checked. The flow rate in the outer lane v_3 is 4,753 – 3,141 = 1,612 pc/h. The average flow rate in Lanes 1 and 2 is 3,141/2 = 1,571 pc/h (rounded). Then:

Is $v_3 > 2{,}700$ pc/h/ln? **No**

Is $v_3 > 1.5 \times 1{,}571 = 2{,}357$ pc/h/ln? **No**

Once again, the predicted lane distribution of arriving vehicles is reasonable, and v_{12} is taken to be 3,141 pc/h.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

Because two off-ramps are involved in this segment, there are several capacity checkpoints:

- Total freeway flow upstream of the first off-ramp (the point at which maximum freeway flow exists),
- Capacity of both off-ramps, and
- Maximum desirable flow rates entering each of the two off-ramp influence areas.

These comparisons are shown in Exhibit 13-22. Note that freeway capacity is based on a freeway with FFS = 60 mi/h. The first ramp capacity is based on a ramp FFS of 40 mi/h and the second on a ramp FFS of 25 mi/h.

Exhibit 13-22
Capacity Checks for Example Problem 2

Item	Capacity (pc/h) Exhibit 13-8, Exhibit 13-10	Demand Flow Rate (pc/h)	Problem?
Freeway flow rate	6,900	5,093	No
First off-ramp	2,000	340	No
Second off-ramp	1,900	566	No
Max. v_{12} first ramp	4,400	3,373	No
Max. v_{12} second ramp	4,400	3,141	No

None of the capacity values are exceeded, so operation of these ramp junctions will be stable, and LOS F does not occur.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-22 and Exhibit 13-2

Because there are two off-ramps, two ramp influence areas are involved, and two ramp influence area densities will be computed.

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

$$D_{R1} = 4.252 + (0.0086 \times 3{,}273) - (0.009 \times 500) = 27.9 \text{ pc/mi/ln}$$

$$D_{R2} = 4.252 + (0.0086 \times 3{,}141) - (0.009 \times 300) = 28.6 \text{ pc/mi/ln}$$

From Exhibit 13-2, both of these ramp influence areas operate very close to the boundary between LOS C and LOS D (28.0 pc/mi/ln). Ramp 1 operates in LOS C, while Ramp 2 operates in LOS D.

While it makes virtually no difference in this case, note that the two ramp influence areas overlap. The influence area of the first off-ramp extends 1,500 ft upstream. The influence area of the second off-ramp also extends 1,500 ft upstream. Given that the ramps are only 750 ft apart, the second ramp influence area overlaps the first for 750 ft (immediately upstream of the first diverge point). Normally, the worst of the two levels of service would be applied to this 750-ft overlap. In this case, the levels of service are the same. Indeed, the predicted densities are virtually equal, so the impact of the overlap is minimal, and the predicted values are not really affected.

Step 5: Compute Diverge Area Speeds as Supplemental Information by Using Exhibit 13-12 and Exhibit 13-13

Because these ramps are on a six-lane freeway with an outer lane, it is possible to estimate the speed within each ramp influence area, the speed in the outer lane adjacent to each ramp influence area, and the weighted average of the two.

First Off-Ramp

The speed within the first ramp influence area is computed as follows:

$$D_S = 0.883 + 0.00009 v_R - 0.013 S_{FR}$$

$$D_S = 0.883 + (0.00009 \times 3{,}273) - (0.013 \times 40) = 0.394$$

$$S_R = FFS - (FFS - 42) D_S = 60 - (60 - 42) \times 0.394 = 52.9 \text{ mi/h}$$

The flow rate in the outer lane (v_{OA}) is 5,093 – 3,273 = 1,820 pc/h/ln. The average speed in this outer lane is computed as follows:

$$S_O = 1.097 FFS - 0.0039\,(v_{OA} - 1{,}000)$$

$$S_O = (1.097 \times 60) - 0.0039 \times (1{,}820 - 1{,}000) = 62.6 \text{ mi/h}$$

The average speed in Lane 3 is predicted to be slightly higher than the FFS of the freeway. This is not uncommon, since through vehicles at higher speeds use Lane 3 to avoid congestion in the ramp influence area. The average speed across all lanes, however, should not be higher than the FFS. In this case, the average speed across all lanes is computed as follows:

$$S = \frac{3{,}273 + (1{,}820 \times 1)}{\left(\frac{3{,}273}{52.9}\right) + \left(\frac{1{,}820 \times 1}{62.0}\right)} = 56.0 \text{ mi/h}$$

This result is, as expected, less than the FFS of the freeway.

Second Off-Ramp

The speed in the second ramp influence area is computed as follows:

$$D_S = 0.883 + (0.00009 \times 566) - (0.013 \times 25) = 0.609$$

$$S_R = 60 - (60 - 42) \times 0.609 = 49.0 \text{ mi/h}$$

Lane 3 has a demand flow rate of 4,753 – 3,141 = 1,612 pc/h/ln. The average speed in this outer lane is computed as follows:

$$S_O = (1.097 \times 60) - 0.0039 \times (1{,}612 - 1{,}000) = 63.4 \text{ mi/h}$$

The average speed across all freeway lanes is

$$S = \frac{3{,}141 + (1{,}612 \times 1)}{\left(\frac{3{,}141}{49.0}\right) + \left(\frac{1{,}612 \times 1}{63.4}\right)} = 53.1 \text{ mi/h}$$

Discussion

The speed results in this case are interesting. While densities are similar for both ramps, the density is somewhat higher and the speed somewhat lower in the second influence area. This is primarily the result of a shorter deceleration lane and a lower ramp FFS (25 mi/h versus 40 mi/h). In both cases, the average speed in the outer lane is higher than the FFS, which applies as an average across all lanes.

Since the operation is stable, there is no special concern here, short of a significant increase in demand flows. LOS is technically D but falls just over the LOS C boundary. This is a case in which the step-function LOS assigned may imply an operation poorer than actually exists. It emphasizes the importance of knowing not only the LOS but also the value of the service measure that produces it.

EXAMPLE PROBLEM 3: ONE-LANE ON-RAMP FOLLOWED BY A ONE-LANE OFF-RAMP ON AN EIGHT-LANE FREEWAY

The Facts

The following information is available concerning this pair of ramps to be analyzed:

- Eight-lane freeway with an FFS of 65 mi/h
- One-lane, right-hand on-ramp with an FFS of 30 mi/h
- One-lane, right-hand off-ramp with an FFS of 25 mi/h
- Distance between ramps = 1,300 ft
- Acceleration lane on Ramp 1 = 260 ft
- Deceleration lane on Ramp 2 = 260 ft
- Level terrain on freeway and both ramps
- 10% trucks, no RVs on freeway and off-ramp
- 5% trucks, no RVs on on-ramp
- Freeway flow rate (upstream of first ramp) = 5,500 veh/h
- On-ramp flow rate = 400 veh/h
- Off-ramp flow rate = 600 veh/h
- PHF = 0.90
- Drivers are regular commuters

Comments

As with previous example problems, the conversion of demand volumes to flow rates requires adjustment factors selected from Chapter 11, Basic Freeway

Segments. All pertinent information is given, and no default values will be applied.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

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

Three demand volumes must be converted to flow rates under equivalent ideal conditions: the freeway volume immediately upstream of the first ramp junction, the first ramp volume, and the second ramp volume. Because the freeway segment under study has level terrain, the value of E_T will be 1.5 for all volumes. Because the drivers are regular commuters, the driver population factor, f_p, is 1.00.

Then, for the freeway demand volume:

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

$$v_F = \frac{5{,}500}{0.90 \times 0.952 \times 1.00} = 6{,}419 \text{ pc/h}$$

For the on-ramp demand volume:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1)} = 0.976$$

$$v_{R1} = \frac{400}{0.90 \times 0.976 \times 1.00} = 455 \text{ pc/h}$$

For the off-ramp demand volume:

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

$$v_{R2} = \frac{600}{0.90 \times 0.952 \times 1.00} = 700 \text{ pc/h}$$

In the remaining computations, these converted demand flow rates are used as input values.

Step 2: Compute Demand Flow in Lanes 1 and 2 Immediately Upstream of the Two Ramp Influence Areas by Using Equation 13-2 and Exhibit 13-6 for the On-Ramp and Equation 13-8 and Exhibit 13-7 for the Off-Ramp

Once again, the situation involves a pair of adjacent ramps. Each ramp must consider the potential impact of the other on its operations. Because the ramps are on an eight-lane freeway (four lanes in each direction), Exhibit 13-6 and Exhibit 13-7 indicate that each ramp is considered as if it were isolated.

First Ramp (On-Ramp)

Equation 13-2 and Exhibit 13-6 apply to on-ramps. Exhibit 13-6 presents two possible equations for use in estimating v_{12} on the basis of the value of v_F/S_{FR}. In this case, the value is 6,419/30 = 210.6 > 72. Therefore, Equation 13-5 is used, giving the following:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.2178 - 0.000125 v_R$$

$$P_{FM} = 0.2178 - (0.000125 \times 455) = 0.161$$

$$v_{12} = 6{,}419 \times 0.161 = 1{,}033 \text{ pc/h}$$

Because the eight-lane freeway includes two outer lanes in each direction, the reasonableness of this prediction must be checked. The average flow per lane in Lanes 1 and 2 is 1,033/2 = 517 pc/h/ln (rounded). The flow in the two outer lanes, Lanes 3 and 4, is 6,419 – 1,033 = 5,386 pc/h. The average flow per lane in Lanes 3 and 4 is, therefore, 5,386/2 = 2,693 pc/h/ln. Then:

Is v_{av34} > 2,700 pc/h/ln? **No**

Is v_{av34} > 1.5 × 517 = 776 pc/h/ln? **Yes**

The predicted lane distribution, therefore, is not reasonable. Too many vehicles are placed in the two outer lanes compared with Lanes 1 and 2. Equation 13-19 is used to produce a more reasonable distribution:

$$v_{12a} = \left(\frac{v_F}{2.50}\right) = \left(\frac{6{,}419}{2.50}\right) = 2{,}568 \text{ pc/h}$$

On the basis of this adjusted value, the number of vehicles now assigned to the two outer lanes is 6,419 – 2,568 = 3,851 pc/h.

Second Ramp (Off-Ramp)

Equation 13-8 and Exhibit 13-7 apply to off-ramps. Exhibit 13-7 shows that the value of P_{FD} for off-ramps on eight-lane freeways is a constant: 0.436. As the methodology is based on regression analysis of a database, the recommendation of a constant reflects a small sample size in that database. Note also that the freeway flow approaching the second ramp is the sum of the freeway flow approaching the first ramp and the on-ramp flow that is now also on the freeway, or 6,419 + 455 = 6,874 pc/h. The flow rate in Lanes 1 and 2 is now easily computed by using Equation 13-8:

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

$$v_{12} = 700 + (6{,}874 - 700) \times 0.436 = 3{,}392 \text{ pc/h}$$

Because there are two outer lanes on this eight-lane freeway, the reasonableness of this estimate must be checked. The average flow per lane in Lanes 1 and 2 is 3,392/2 = 1,696 pc/h/ln. The total flow in Lanes 3 and 4 of the freeway is 6,874 – 3,392 = 3,482 pc/h, or an average flow rate per lane of 3,482/2 = 1,741 pc/h/ln.

Is v_{av34} > 2,700 pc/h/ln? **No**

Is $v_{av34} > 1.5 \times 1{,}696 = 2{,}544$ pc/h/ln? **No**

Therefore, the estimated value of v_{12} is deemed reasonable and is carried forward in the computations.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

Because there are two ramps in this segment, there are five capacity checkpoints to consider:

- The freeway flow rate at its maximum point—which in this case is between the on- and off-ramp, since this is the only location where both on- and off-ramp vehicles are on the freeway.
- The capacity of the on-ramp.
- The capacity of the off-ramp.
- The maximum desirable flow entering the on-ramp influence area.
- The maximum desirable flow entering the off-ramp influence area.

These comparisons are shown in Exhibit 13-23. The capacity of the freeway is based on an eight-lane freeway with an FFS of 65 mi/h. The capacity of the on-ramp is based on an FFS of 30 mi/h, and the capacity of the off-ramp is based on an FFS of 25 mi/h.

Exhibit 13-23
Capacity Checks for Example Problem 3

Item	Capacity (pc/h) Exhibit 13-8, Exhibit 13-10	Demand Flow Rate (pc/h)	Problem?
Freeway flow rate	9,400	6,874	No
First on-ramp	1,900	345	No
Second off-ramp	1,900	700	No
Max. v_{R12} first ramp	4,600	2,568 + 455 = 3,023	No
Max. v_{12} second ramp	4,400	3,392	No

There are no capacity concerns, since all demands are well below the associated capacities or maximum desirable values. LOS F is not present in any part of this segment, and operations are expected to be stable.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-21, Equation 13-22, and Exhibit 13-2

Equation 13-21 is used to find the density in the first on-ramp influence area:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

$$D_R = 5.475 + (0.00734 \times 455) + (0.0078 \times 2{,}568) - (0.00627 \times 260) = 27.2 \text{ pc/mi/ln}$$

Equation 13-22 is used to find the density in the second off-ramp influence area:

$$D_R = 4.252 + 0.0086 v_{12} - 0.009 L_D$$

$$D_R = 4.252 + (0.0086 \times 3{,}391) - (0.009 \times 260) = 31.1 \text{ pc/mi/ln}$$

From Exhibit 13-2, both of these ramp influence areas operate very close to the boundary between LOS C and LOS D (28 pc/mi/ln). Ramp 1 operates in LOS C, while Ramp 2 operates in LOS D.

Because the on-ramp influence area extends 1,500 ft downstream, the off-ramp influence area extends 1,500 ft upstream, and the two ramps are only 1,300 ft apart, the distance between the ramps is included in both. Therefore, the more pessimistic prediction of LOS D for the off-ramp governs the operation. Oddly, the additional 200 ft of the off-ramp influence area is actually upstream of the on-ramp, and the additional 200 ft of the on-ramp influence area is downstream of the off-ramp.

Step 5: Compute Merge and Diverge Area Speeds as Supplemental Information by Using Exhibit 13-11 and Exhibit 13-12

Because of the eight-lane freeway, speeds should be estimated for the two ramp influence areas, for the outer lanes (Lanes 3 and 4) adjacent to the ramp influence area, and for all vehicles—the weighted average of the other two speeds.

First Ramp (On-Ramp)

Equations for estimation of average speed in an on-ramp influence area and in outer lanes adjacent to it are taken from Exhibit 13-11.

$$M_S = 0.321 + 0.0039e^{(v_{R12}/1{,}000)} - 0.002(L_A S_{FR}/1{,}000)$$

$$M_S = 0.321 + 0.0039e^{(3{,}032/1{,}000)} - 0.002(260/30) = 0.385$$

$$S_R = FFS - (FFS - 42)M_S = 65 - (65 - 42) \times 0.385 = 56.2 \text{ mi/h}$$

Since the average outer lane demand flow rate is 3,851/2 = 1,926 pc/h/ln, which is greater than 500 pc/h/ln and less than 2,300 pc/h/ln, the outer speed is estimated as follows:

$$S_O = FFS - 0.0036\,(v_{OA} - 500)$$

$$S_O = 65 - 0.0036(1{,}926 - 500) = 59.9 \text{ mi/h}$$

The weighted average speed of all vehicles is

$$S = \frac{3{,}032 + (1{,}926 \times 2)}{\left(\frac{3{,}032}{56.2}\right) + \left(\frac{1{,}926 \times 2}{59.9}\right)} = 58.2 \text{ mi/h}$$

Second Ramp (Off-Ramp)

For off-ramps, equations for estimation of average speed are drawn from Exhibit 13-12. At the second ramp, the flow in Lanes 1 and 2 has been computed as 3,392 pc/h or 1,696 pc/h/ln, while the flow in Lanes 3 and 4 is 3,482 pc/h, or 1,741 pc/h/ln. Then

$$D_S = 0.883 + 0.00009v_R - 0.013S_{FR}$$

$$D_S = 0.883 + (0.00009 \times 700) - (0.013 \times 25) = 0.621$$

$$S_R = FFS - (FFS - 42)D_S$$

$$S_R = 65 - (65 - 42) \times 0.621 = 50.7 \text{ mi/h}$$

Because the average flow in the outer lanes is greater than 1,000 pc/h/ln, the average speed of vehicles in the outer lanes (Lanes 3 and 4) is as follows:

$$S_O = 1.097 FFS - 0.0039\left(v_{OA} - 1{,}000\right)$$

$$S_O = (1.097 \times 65) - 0.0039\left(1{,}741 - 1{,}000\right) = 68.4 \text{ mi/h}$$

The weighted average speed of all vehicles is

$$S = \frac{3{,}392 + (1{,}741 \times 2)}{\left(\frac{3{,}392}{50.7}\right) + \left(\frac{1{,}741 \times 2}{68.4}\right)} = 58.3 \text{ mi/h}$$

Discussion

As noted previously, between the ramps, the influence areas of both ramps fully overlap. Since a higher density is predicted for the off-ramp influence area, and LOS D results, this density should be applied to the entire area between the two ramps.

The speed results are also interesting. The slower speeds within the off-ramp influence area will also control the overlap area. On the other hand, the speed results indicate a higher average speed for all vehicles associated with the off-ramp than the speed associated with the on-ramp. This is primarily due to the much larger disparity between speeds within the ramp influence area and in outer lanes when the off-ramp is considered. The speed differential is more than 20 mi/h for the off-ramp, as opposed to a little more than 3 mi/h for the on-ramp. This is not entirely unexpected. At diverge junctions, vehicles in outer lanes tend to face less turbulence than those in outer lanes near merge junctions. All off-ramp vehicles must be in Lanes 1 and 2 for some distance before exiting the freeway. On-ramp vehicles, on the other hand, can execute as many lane changes as they wish—consistent with safety and sanity—and more of them may wind up in outer lanes within 1,500 ft of the junction point.

Thus, the total operation of this two-ramp segment is expected to be LOS D, with speeds of approximately 50 mi/h in Lanes 1 and 2 and approximately 70 mi/h in Lanes 3 and 4.

EXAMPLE PROBLEM 4: SINGLE-LANE, LEFT-HAND ON-RAMP ON A SIX-LANE FREEWAY

The Facts

- One-lane, left-side on-ramp on a six-lane freeway (three lanes in each direction)
- Freeway demand volume upstream of ramp = 4,000 veh/h
- On-ramp demand volume = 500 veh/h
- 15% trucks, no RVs on freeway
- 5% trucks, no RVs on ramp
- Freeway FFS = 65 mi/h
- Ramp FFS = 30 mi/h

- Acceleration lane = 820 ft
- Level terrain on freeway and ramp
- Drivers are regular commuters

Comments

This is a special application of the ramp analysis methodology presented in this chapter. For left-hand ramps, the flow rate in Lanes 1 and 2 (v_{12}) is initially computed as if it were a right-hand ramp. Exhibit 13-16 is then used to convert this result to an estimate of the flow in Lanes 2 and 3 (v_{23}), since these are the two leftmost lanes that will be involved in the merge. In effect, the ramp influence area is, in this case, Lanes 3 and 4 and the acceleration lane for a distance of 1,500 ft downstream of the merge point.

Step 1: Convert Demand Volumes to Flow Rates Under Equivalent Ideal Conditions by Using Equation 13-1

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

From Chapter 11, Basic Freeway Segments, the passenger car equivalent E_T for trucks in level terrain is 1.5. The driver population adjustment factor f_p for regular commuters is 1.00.

For the freeway demand volume:

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

$$f_{HV} = \frac{1}{1 + 0.15(1.5 - 1)} = 0.930$$

$$v_F = \frac{4{,}000}{0.90 \times 0.93 \times 1.00} = 4{,}779 \text{ pc/h}$$

For the ramp demand volume:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1)} = 0.976$$

$$v_R = \frac{500}{0.90 \times 0.976 \times 1.00} = 569 \text{ pc/h}$$

Step 2: Compute Demand Flow in Lanes 2 and 3 Immediately Upstream of the Ramp Influence Area by Using Equation 13-2 and Exhibit 13-6

To estimate flow in the two left lanes, the flow normally expected in Lanes 1 and 2 for a similar right-hand ramp must first be computed. From Exhibit 13-6, for an isolated on-ramp on a six-lane freeway, Equation 13-4 is used:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.5775 + 0.000028 L_A$$

$$P_{FM} = 0.5775 + (0.000028 \times 820) = 0.600$$

$$v_{12} = 4{,}779 \times 0.600 = 2{,}867 \text{ pc/h}$$

From Exhibit 13-16, the adjustment factor applied to this result to find the estimated flow rate in Lanes 2 and 3 is 1.12. Therefore:

$$v_{23} = 2{,}867 \times 1.12 = 3{,}211 \text{ pc/h}$$

While, strictly speaking, the reasonableness criteria for lane distribution do not apply to left-hand ramps, they can be applied very approximately. In this case, the single "outer lane" (which is now Lane 1) would have a flow rate of 4,779 – 3,211 = 1,568 pc/h. This is not greater than 2,700 pc/h/ln, nor is it greater than 1.5 times the average flow in Lanes 2 and 3 (1.5 × 3,211/2 = 2,408 pc/h/ln). Thus, even if the reasonableness criteria were approximately applied in this case, no violation would exist.

The remaining computations proceed for the left-hand ramp, with the substitution of v_{34} for v_{12} in all algorithms used.

Step 3: Check Capacities by Using Exhibit 13-8 and Exhibit 13-10

For this case, there are three simple checkpoints:

- The principal capacity checkpoint is the total demand flow rate downstream of the merge, 4,779 + 569 = 5,348 pc/h. From Exhibit 13-8, for a six-lane freeway with an FFS of 65 mi/h, the capacity is 7,050 pc/h, well over the demand flow rate.
- The ramp roadway capacity should also be checked by using Exhibit 13-10. For a single-lane ramp with an FFS of 30 mi/h, the capacity is 1,900 pc/h, which is much greater than the demand flow rate of 569 pc/h.
- Finally, the maximum flow entering the ramp influence area should be checked. In this case, a left-hand ramp, the total flow entering the ramp influence area is the freeway flow remaining in Lanes 2 and 3 plus the ramp flow rate. Thus, the total flow entering the ramp influence area is 3,211 + 569 = 3,780 pc/h, which is lower than the maximum desirable flow rate of 4,600 pc/h, shown in Exhibit 13-8.

Thus, there are no capacity problems at this merge point, and stable operations are expected. LOS F will not result from the stated conditions.

Step 4: Compute Densities and Find Levels of Service by Using Equation 13-21 and Exhibit 13-2

The density in the ramp influence area is found by using Equation 13-21, except v_{23} replaces v_{12} because of the left-hand ramp placement:

$$D_S = 5.475 + 0.00734 v_R + 0.0078 v_{23} - 0.00627 L_A$$

$$D_S = 5.475 + (0.00734 \times 569) + (0.0078 \times 3{,}211) - (0.00627 \times 820) = 29.6 \text{ pc/mi/ln}$$

From Exhibit 13-2, this is LOS D.

Step 5: Compute Merge and Diverge Area Speeds as Supplemental Information by Using Exhibit 13-11 and Exhibit 13-13

The speed estimation algorithms were calibrated for right-hand ramps, and the estimation algorithms for "outer lane(s)" assume that these are the leftmost lanes. Thus, for a left-hand ramp, these computations must be considered approximate at best.

By using the equations in Exhibit 13-11 and Exhibit 13-13, the following results are obtained:

$$M_S = 0.321 + 0.0039 e^{(3{,}780/1{,}000)} - 0.002(820 \times 30 / 1{,}000) = 0.443$$

$$S_R = 65 - (65 - 42) \times 0.443 = 54.8 \text{ mi/h}$$

$$S_O = 65 - 0.0036\ (1{,}568 - 500) = 61.2 \text{ mi/h}$$

$$S = \frac{3{,}780 + (1{,}568 \times 1)}{\left(\frac{3{,}780}{54.8}\right) + \left(\frac{1{,}568 \times 1}{61.2}\right)} = 56.5 \text{ mi/h}$$

While traffic in the outer lane is predicted to travel somewhat faster than traffic in the lanes in the ramp influence area (which includes the acceleration lane), the approximate nature of the speed result for left-hand ramps makes it difficult to draw any firm conclusions concerning speed behavior.

Discussion

This example problem is typical of the way the situations in the Special Cases section are treated. Modifications as specified are applied to the standard algorithms used for single-lane, right-hand ramp junctions. In this case, operations are acceptable, but in LOS D—though not far from the LOS C boundary. Because the left-hand lanes are expected to carry freeway traffic flowing faster than right-hand lanes, right-hand ramps are normally preferable to left-hand ramps when they can be provided without great difficulty.

EXAMPLE PROBLEM 5: SERVICE FLOW RATES AND SERVICE VOLUMES FOR AN ISOLATED ON-RAMP ON A SIX-LANE FREEWAY

The Facts

The following facts have been established for this situation:

- Single-lane, right-hand on-ramp with an FFS of 40 mi/h
- Six-lane freeway (three lanes in each direction) with an FFS of 70 mi/h
- Level terrain for freeway and ramp
- 12% trucks, 3% RVs on freeway
- 5% trucks, 2% RVs on ramp
- Peak hour factor = 0.87
- Drivers are regular users of the facility
- Acceleration lane = 1,000 ft

Comments

This example illustrates the computation of service flow rates and service volumes for a ramp–freeway junction. The case selected is relatively straightforward to avoid cluttering the illustration with extraneous complications that have been addressed in other example problems.

Two approaches will be demonstrated:

1. The ramp demand flow rate will be stated as a fixed percentage of the arriving freeway flow rate. The service flow rates and service volumes are expressed as arriving freeway flow rates that result in the threshold densities within the ramp influence area that define the limits of the various levels of service. For this computation, the ramp flow is set at 10% of the approaching freeway flow rate.
2. A fixed freeway demand flow rate will be stated, with service flow rates and service volumes expressed as ramp demand flow rates that result in the threshold densities within the ramp influence area that define the limits of the various levels of service. For this computation, the approaching freeway flow rate is set at 4,000 veh/h.

For LOS E, density does not define the limiting value of service flow rate, which is analogous to capacity for ramp–freeway junctions. It is defined as the flow that results in capacity being reached on the downstream freeway segment or ramp roadway.

Since all algorithms in this methodology are calibrated for passenger cars per hour under equivalent ideal conditions, initial computations are made in those terms. Results are then converted to service flow rates by using the appropriate heavy vehicle and driver population adjustment factors. Service flow rates are then converted to service volumes by multiplying by the peak hour factor.

From Exhibit 13-2, the following densities define the limits of LOS A–D:

LOS A	10 pc/mi/ln
LOS B	20 pc/mi/ln
LOS C	28 pc/mi/ln
LOS D	35 pc/mi/ln

From Exhibit 13-8 and Exhibit 13-10, capacity (or the threshold for LOS E) occurs when the downstream freeway flow rate reaches 7,200 pc/h (FFS = 70 mi/h) or when the ramp flow rate reaches 2,000 pc/h (ramp FFS = 40 mi/h).

Case 1: Ramp Demand Flow Rate = 0.10 Freeway Demand Flow Rate

Equation 13-21 defines the density in an on-ramp influence area as follows:

$$D_R = 5.475 + 0.00734 v_R + 0.0078 v_{12} - 0.00627 L_A$$

In this case

v_R = 0.10 v_F

L_A = 1,000 ft

Equation 13-2 and Exhibit 13-6 give the following:

$$v_{12} = v_F \times P_{FM}$$

$$P_{FM} = 0.5775 + 0.000028 L_A$$

$$P_{FM} = 0.5775 + (0.000028 \times 1{,}000) = 0.6055$$

$$v_{12} = 0.6055 v_F$$

Substitution of these values into Equation 13-21 gives

$$D_R = 5.475 + (0.00734 \times 0.10 v_F) + (0.0078 \times 0.6055 v_F) - (0.00627 \times 1{,}000)$$

$$D_R = 5.475 + 0.000734 v_F + 0.00472 v_F - 6.27$$

$$D_R = 0.005454 v_F - 0.795$$

$$v_F = \frac{D_R + 0.795}{0.005454}$$

This equation can now be solved for threshold values of v_F for LOS A through D by using the appropriate threshold values of density. The results will be in terms of service flow rates under equivalent ideal conditions:

$$v_F(\text{LOS A}) = \frac{10 + 0.795}{0.005454} = 1{,}979 \text{ pc/h}$$

$$v_F(\text{LOS B}) = \frac{20 + 0.795}{0.005454} = 3{,}813 \text{ pc/h}$$

$$v_F(\text{LOS C}) = \frac{28 + 0.795}{0.005454} = 5{,}280 \text{ pc/h}$$

$$v_F(\text{LOS D}) = \frac{35 + 0.795}{0.005454} = 6{,}563 \text{ pc/h}$$

At capacity, the limiting flow rate occurs when the downstream freeway segment is 7,200 pc/h. If the ramp flow rate is 0.10 of the approaching freeway flow rate, then

$$v_{FO} = 7{,}200 = v_F + 0.10 v_F = 1.10 v_F$$

$$v_F(\text{LOS E}) = \frac{7{,}200}{1.10} = 6{,}545 \text{ pc/h}$$

This must be checked to ensure that the ramp flow rate (0.10 × 6,545 = 655 pc/h) does not exceed the ramp capacity of 2,000 pc/h. Since it does not, the computation stands.

Note, however, that the LOS E (capacity) threshold is lower than the LOS D threshold. This indicates that LOS D operation cannot be achieved at this location. Before densities reach the 35 pc/h/ln threshold for LOS D, the capacity of the merge junction has been reached. Thus, there is no service flow rate or service volume for LOS D.

The computed values, as noted, are in terms of passenger cars per hour under equivalent ideal conditions. To convert these to service flow rates in vehicles per hour under prevailing conditions, they must be multiplied by the heavy vehicle adjustment factor and the driver population factor. The approaching freeway flow includes 12% trucks and 3% RVs. For level terrain (Chapter 11, Basic Freeway Segments), $E_T = 1.5$ and $E_R = 1.2$. Then

$$f_{HV} = \frac{1}{1+0.12(1.5-1)+0.03(1.2-1)} = 0.938$$

The driver population factor f_p for regular facility users is 1.00. Service volumes are obtained by multiplying service flow rates by the specified PHF, 0.87. These computations are illustrated in Exhibit 13-24.

Exhibit 13-24
Illustrative Service Flow Rates and Service Volumes Based on Approaching Freeway Demand

LOS	Service Flow Rate, Ideal Conditions (pc/h)	Service Flow Rate, Prevailing Conditions (SF) (veh/h)	Service Volume (SV) (veh/h)
A	1,979	1,979 × 0.938 × 1 = 1,856	1,856 × 0.87 = 1,615
B	3,813	3,813 × 0.938 × 1 = 3,577	3,577 × 0.87 = 3,112
C	5,280	5,280 × 0.938 × 1 = 4,953	4,953 × 0.87 = 4,309
D	NA	NA	NA
E	6,545	6,545 × 0.938 × 1 = 6,139	6,139 × 0.87 = 5,341

The service flow rates and service volumes shown in Exhibit 13-24 are stated in terms of the approaching freeway demand.

Case 2: Approaching Freeway Demand Volume = 4,000 veh/h

In this case, the approaching freeway demand will be held constant, and service flow rates and service volumes will be stated in terms of the ramp demand that can be accommodated at each LOS.

Since the freeway demand is stated in terms of an hourly volume in mixed vehicles per hour, it will be converted to passenger cars per hour under equivalent ideal conditions for use in the algorithms of this methodology:

$$v_F = \frac{V_F}{PHF \times f_{HV} \times f_p} = \frac{4{,}000}{0.87 \times 0.938 \times 1} = 4{,}902 \text{ pc/h}$$

The density is estimated by using Equation 13-20, and the variable P_{FM}—which is not dependent on v_R—remains 0.6055 as in Case 1. With a fixed value of freeway demand:

$$v_{12} = 0.6055 \times 4{,}902 = 2{,}968 \text{ pc/h}$$

Then, by using Equation 13-21:

$$D_R = 5.475 + 0.00734 v_R + (0.0078 \times 2{,}968) - (0.00627 \times 1{,}000)$$

$$D_R = 22.355 + 0.00734 v_R$$

$$v_R = \frac{D_R - 22.355}{0.00734}$$

It is apparent from this equation that neither LOS A ($D_R = 10$ pc/mi/ln) nor LOS B ($D_R = 20$ pc/mi/ln) can be achieved with a fixed freeway demand flow of 4,902 pc/h.

For LOS C and D:

$$v_R(\text{LOS C}) = \frac{28 - 22.355}{0.00734} = 769 \text{ pc/h}$$

$$v_R(\text{LOS D}) = \frac{35 - 22.355}{0.00734} = 1{,}723 \text{ pc/h}$$

Capacity, the limit of LOS E, occurs when the downstream freeway flow reaches 7,200 pc/h. With a fixed freeway demand:

$$v_{FO} = 7{,}200 = 4{,}902 + v_R$$

$$v_R(\text{LOS E}) = 7{,}200 - 4{,}902 = 2{,}298 \text{ pc/h}$$

This, however, violates the capacity of the ramp roadway, which is 2,000 pc/h. Thus, the limiting ramp flow rate for LOS E is set at 2,000 pc/h.

As in Case 1, these values are all stated in terms of passenger cars per hour under equivalent ideal conditions. They are converted to service flow rates by multiplying by the appropriate heavy vehicle and driver population adjustment factors. Since the ramp has a composition different from that of the approaching freeway flow, its adjustment must be recomputed:

$$f_{HV} = \frac{1}{1 + 0.05(1.5 - 1) + 0.02(1.2 - 1)} = 0.972$$

Service flow rates are converted to service volumes by multiplying by the peak hour factor. These computations are illustrated in Exhibit 13-25.

Exhibit 13-25
Illustrative Service Flow Rates and Service Volumes Based on a Fixed Freeway Demand

LOS	Service Flow Rate, Ideal Conditions (pc/h)	Service Flow Rate, Prevailing Conditions (SF) (veh/h)	Service Volume (SV) (veh/h)
A	NA	NA	NA
B	NA	NA	NA
C	769	769 × 0.972 × 1 = 747	747 × 0.87 = 650
D	1,723	1,723 × 0.972 × 1 = 1,675	1,675 × 0.87 = 1,457
E	2,000	2,000 × 0.972 × 1 = 1,944	1,944 × 0.87 = 1,691

These service flow rates and service volumes are based on a constant upstream arriving freeway demand and are stated in terms of limiting on-ramp demands for that condition.

Discussion

As this illustration shows, many considerations are involved in estimating service flow rates and service volumes for ramp–freeway junctions, not the least of which is specifying how such values should be defined. The concept of service flow rates and service volumes at specific ramp–freeway junctions is of limited utility. Since many of the details that affect the estimates will not be determined until final designs are prepared, operational analysis of the proposed design may be more appropriate.

Case 2 could have applications in considering how to time ramp meters. Appropriate limiting ramp flows can be estimated by using the same approach as for service volumes and service flow rates.

5. REFERENCES

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

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