Three lengths are involved in analyzing a weaving segment:

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

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

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

In the Chapter 12 weaving methodology, the value of $L_{wMAX}$ depends on a number of factors, including the split of component flows, demand flows, and other traffic factors. A weaving configuration could therefore qualify as a weaving segment in some analysis periods and as separate merge, diverge, and possibly basic segments in others.

In segmenting the freeway facility for analysis, merge, diverge, and weaving segments are identified as illustrated in Exhibit 10-12 and Exhibit 10-13. All segments not qualifying as merge, diverge, or weaving segments are basic freeway segments.
However, a long basic freeway section may have to be divided into multiple segments. This situation occurs when there is a sharp break in terrain within the section. For example, a 5-mi section may have a constant demand and a constant number of lanes. If there is a 2-mi level terrain portion followed by a 4% grade that is 3 mi long, then the level terrain portion and the specific grade portion would be established as two separate, consecutive basic freeway segments.

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

Traffic counts taken at each entrance to and exit from the defined freeway facility (including the mainline entrance and mainline exit) for each time interval serve as inputs to the methodology. While entrance counts are considered to represent the current entrance demands for the freeway facility (provided that there is not a queue on the freeway entrance), the exit counts may not represent the current exit demands for the freeway facility because of congestion within the defined facility.

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

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

Demand estimation is needed if the methodology uses actual freeway counts. If demand flows are known or can be projected, they are used directly without modification.

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

The procedure sums the freeway entrance demands along the entire directional freeway facility, including the entering mainline segment, and compares this sum with the sum of freeway exit counts along the directional freeway facility, including the departing mainline segment. This procedure is repeated for each time interval. The ratio of the total facility entrance counts to total facility exit counts is called the *time interval scale factor* and should approach 1.00 when the freeway exit counts are, in fact, freeway exit demands.

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

Equation 10-6 and Equation 10-7 summarize this process.
of time or at frequent intervals. Crawl speed is the maximum sustained speed that trucks can maintain on an extended upgrade of a given percent. If the grade is long enough, trucks will be forced to decelerate to the crawl speed, which they can maintain for extended distances. Appendix A contains truck-performance curves illustrating crawl speed and length of grade.

- **Mountainous terrain:** Any combination of grades and horizontal and vertical alignment that causes heavy vehicles to operate at crawl speed for significant distances or at frequent intervals.

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

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

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>PCE by Type of Terrain</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Level</td>
</tr>
<tr>
<td>Trucks and buses, $E_T$</td>
<td>1.5</td>
</tr>
<tr>
<td>RVs, $E_R$</td>
<td>1.2</td>
</tr>
</tbody>
</table>

**Equivalents for Specific Upgrades**

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

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

Exhibit 11-11 and Exhibit 11-12 give values of $E_T$ for trucks and buses and $E_R$ for RVs, respectively. These factors vary with the percent of grade, length of grade, and the proportion of heavy vehicles in the traffic stream. Maximum values occur when there are only a few heavy vehicles in the traffic stream. The equivalents decrease as the number of heavy vehicles increases because these vehicles tend to form platoons. Because heavy vehicles have more uniform operating characteristics, fewer large gaps are created in the traffic stream when they platoon, and the impact of a single heavy vehicle in a platoon is less severe than that of a single heavy vehicle in a stream of primarily passenger cars. The aggregate impact of heavy vehicles on the traffic stream, however, increases as numbers and percentages of heavy vehicles increase.
The grade length should include 25% of the length of the vertical curves at the start and end of the grade.

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

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

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

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

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

<table>
<thead>
<tr>
<th>Upgrade (%)</th>
<th>Length (mi)</th>
<th>2%</th>
<th>4%</th>
<th>5%</th>
<th>6%</th>
<th>8%</th>
<th>10%</th>
<th>15%</th>
<th>20%</th>
<th>≥25%</th>
</tr>
</thead>
<tbody>
<tr>
<td>≤2</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;2–3</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;3–4</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;4–5</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;5–6</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&gt;6</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

The methodology of this chapter is most often used to estimate the capacity and LOS of freeway weaving segments. The steps are most easily applied in the operational analysis mode, that is, all traffic and roadway conditions are specified, and a solution for the capacity (and $v/c$ ratio) is found along with an expected LOS. Other types of analysis, however, are possible.

DEFAULT VALUES

An NCHRP report (10) provides a comprehensive presentation of potential default values for uninterrupted-flow facilities. Default values for freeways are summarized in Chapter 11, Basic Freeway Segments. These defaults cover the key characteristics of PHF and percentage of heavy vehicles. Recommendations are based on geographical region, population, and time of day. All general freeway default values may be applied to the analysis of weaving segments in the absence of field data or projected conditions.

There are many specific variables related to weaving segments. It is, therefore, virtually impossible to specify default values of such characteristics as length, width, configuration, and balance of weaving and nonweaving flows. Weaving segments are a detail of the freeway design and should therefore be treated only with the specific characteristics of the segment known or projected. Small changes in some of these variables can and do yield significant changes in the analysis results.

TYPES OF ANALYSIS

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

Operational Analysis

The methodology of this chapter is most easily applied in the operational analysis mode. In this application, all weaving demands and geometric characteristics are known, and the output of the analysis is the expected LOS and the capacity of the segment. Secondary outputs include the average speed of component flows, the overall density in the segment, and measures of lane-changing activity.

Design Analysis

In design applications, the desired output is the length, width, and configuration of a weaving segment that will sustain a target LOS for given demand flows. This application is best accomplished by iterative operational analyses on a small number of candidate designs.

Generally, there is not a great deal of flexibility in establishing the length and width of a segment, and only limited flexibility in potential configurations. The location of intersecting facilities places logical limitations on the length of the weaving segment. The number of entry and exit lanes on ramps and the freeway itself limits the number of lanes to, at most, two choices. The entry and exit
design of ramps and the freeway facility also produces a configuration that can generally only be altered by adding or subtracting a lane from an entry or exit roadway. Thus, iterative analyses of candidate designs are relatively easy to pursue, particularly with the use of HCM-replicating software.

**Planning and Preliminary Engineering**

Planning and preliminary engineering applications generally have the same desired outputs as design applications: the geometric design of a weaving segment that can sustain a target LOS for specified demand flows.

In the planning and preliminary design phase, however, demand flows are generally stated as average annual daily traffic (AADT) statistics that must be converted to directional design hour volumes. A number of variables may be unknown (e.g., PHF and percentage of heavy vehicles); these may be replaced by default values.

**Service Flow Rates, Service Volumes, and Daily Service Volumes**

This manual defines three sets of values that are related to LOS boundary conditions:

- \( S_F_i \) = service flow rate for LOS \( i \) (veh/h),
- \( S_V_i \) = service volume for LOS \( i \) (veh/h), and
- \( D_SV_i \) = daily service volume for LOS \( i \) (veh/day).

The **service flow rate** is the maximum rate of flow for a 15-min interval that can be accommodated on a segment while maintaining all operational criteria for LOS \( i \) under prevailing roadway and traffic conditions. The **service volume** is the maximum hourly volume that can be accommodated on a segment while maintaining all operational criteria for LOS \( i \) during the worst 15 min of the hour under prevailing roadway and traffic conditions. The **daily service volume** is the maximum AADT that can be accommodated on a segment while maintaining all operational criteria for LOS \( i \) during the worst 15 min of the peak hour under prevailing roadway and traffic conditions. The service flow rate and service volume are unidirectional values, while the daily service volume is a total two-way volume. In the context of a weaving section, the daily service volume is highly approximate, as it is rare that both directions of a freeway have a weaving segment with similar geometry.

In general, service flow rates are initially computed for ideal conditions and are then converted to prevailing conditions by using Equation 12-23 and the appropriate adjustment factors from Chapter 11, Basic Freeway Segments:

\[
S_F_i = SFI_i \times f_{HV} \times f_p
\]

where

- \( SFI_i \) = service flow rate under ideal conditions (pc/h),
- \( f_{HV} \) = adjustment factor for heavy-vehicle presence (Chapter 11), and
- \( f_p \) = adjustment factor for driver population (Chapter 11).
CHAPTER 13
FREEWAY MERGE AND DIVERGE SEGMENTS

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For \( v_f / S_{IR} \leq 72: \quad P_{FM} = 0.2178 - 0.000125 v_f + 0.01115 (L_A / S_{IR}) \)

For \( v_f / S_{IR} > 72: \quad P_{FM} = 0.2178 - 0.000125 v_f \)

**SELECTING EQUATIONS FOR \( P_{FM} \) FOR SIX-LANE FREEWAYS**

<table>
<thead>
<tr>
<th>Adjacent Upstream Ramp</th>
<th>Subject Ramp</th>
<th>Adjacent Downstream Ramp</th>
<th>Equation(s) Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>On</td>
<td>None</td>
<td>Equation 13-3</td>
</tr>
<tr>
<td>None</td>
<td>On</td>
<td>On</td>
<td>Equation 13-3</td>
</tr>
<tr>
<td>None</td>
<td>On</td>
<td>Off</td>
<td>Equation 13-5 or 13-3</td>
</tr>
<tr>
<td>On</td>
<td>On</td>
<td>None</td>
<td>Equation 13-3</td>
</tr>
<tr>
<td>Off</td>
<td>On</td>
<td>None</td>
<td>Equation 13-4 or 13-3</td>
</tr>
<tr>
<td>On</td>
<td>On</td>
<td>On</td>
<td>Equation 13-3</td>
</tr>
<tr>
<td>On</td>
<td>On</td>
<td>Off</td>
<td>Equation 13-5 or 13-3</td>
</tr>
<tr>
<td>Off</td>
<td>On</td>
<td>On</td>
<td>Equation 13-3</td>
</tr>
<tr>
<td>Off</td>
<td>On</td>
<td>Off</td>
<td>Equation 13-5 or 13-4 or 13-3</td>
</tr>
</tbody>
</table>

Note: * 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:

\[
L_{EQ} = 0.214(v_f + v_R) + 0.444L_A + 52.32S_{FR} - 2.403
\]

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

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

where all terms are as previously defined.

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

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:

\[ v_{12} = v_R + (v_F - v_R)P_{FD} \]

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 through freeway 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

#### Equation 13-9
For four-lane freeways:

\[ P_{FD} = 0.760 - 0.000025v_F - 0.000046v_R \]

#### Equation 13-10
For six-lane freeways:

\[ P_{FD} = 0.717 - 0.000039v_F + 0.604\left(\frac{v_{UL}}{L_{UL}}\right) \]

when \( v_{UL}/L_{UL} \leq 0.2 \)

\[ P_{FD} = 0.616 - 0.000021v_F + 0.124\left(\frac{v_D}{L_{DOWN}}\right) \]

#### Equation 13-11
For eight-lane freeways:

\[ P_{FD} = 0.436 \]

### SELECTING EQUATIONS FOR \( P_{FD} \) FOR SIX-LANE FREeways

<table>
<thead>
<tr>
<th>Adjacent Upstream Ramp</th>
<th>Subject Ramp</th>
<th>Adjacent Downstream Ramp</th>
<th>Equation(s) Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>Off</td>
<td>None</td>
<td>Equation 13-9</td>
</tr>
<tr>
<td>None</td>
<td>Off</td>
<td>On</td>
<td>Equation 13-9</td>
</tr>
<tr>
<td>None</td>
<td>Off</td>
<td>Off</td>
<td>Equation 13-11 or 13-9</td>
</tr>
<tr>
<td>On</td>
<td>Off</td>
<td>None</td>
<td>Equation 13-10 or 13-9</td>
</tr>
<tr>
<td>On</td>
<td>Off</td>
<td>None</td>
<td>Equation 13-9</td>
</tr>
<tr>
<td>On</td>
<td>Off</td>
<td>On</td>
<td>Equation 13-10 or 13-9</td>
</tr>
<tr>
<td>On</td>
<td>Off</td>
<td>Off</td>
<td>Equation 13-11 or 13-9</td>
</tr>
<tr>
<td>Off</td>
<td>Off</td>
<td>On</td>
<td>Equation 13-9</td>
</tr>
<tr>
<td>Off</td>
<td>Off</td>
<td>Off</td>
<td>Equation 13-11 or 13-9</td>
</tr>
</tbody>
</table>

Note: 4 lanes = two lanes in each direction; 6 lanes = three lanes in each direction; 8 lanes = four lanes in each direction.  

When \( v_{UL}/L_{UL} > 0.2 \), use Equation 13-9.  
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:

\[
L_{EQ} = \frac{v_u}{0.071 + 0.000023v_f - 0.000076v_R}
\]

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

\[
L_{EQ} = \frac{v_D}{1.15 - 0.000032v_f - 0.000369v_R}
\]

where all terms are as previously defined.

In cases where Equation 13-12 indicates that Equation 13-10 should be used to determine \( P_{FD} \), but \( v_u/L_{UP} > 0.20 \), Equation 13-9 must be used as a default. This is due to the valid calibration range of Equation 13-10, and the fact that it will yield unreasonable results when \( v_u/L_{UP} \) exceeds 0.20. This will lead to step-function changes in \( P_{FD} \) for values just below or above \( v_u/L_{UP} = 0.20 \).

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.

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.

**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:

---

\textbf{Equation 13-12}

\[
L_{EQ} = \frac{v_u}{0.071 + 0.000023v_f - 0.000076v_R}
\]

\textbf{Equation 13-13}

\[
L_{EQ} = \frac{v_D}{1.15 - 0.000032v_f - 0.000369v_R}
\]
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:

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

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:

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

If $v_3$ is greater than $1.5 \times (v_{12}/2)$, use 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:

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

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:

$$v_{12a} = v_F - 5,400$$
If $v_{w34}$ is greater than $1.5 \times (v_{12}/2)$, use 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:

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.

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.

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:

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

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)

<table>
<thead>
<tr>
<th>FFS (mi/h)</th>
<th>Capacity of Upstream/Downstream Freeway Segment</th>
<th>Max. Desirable Flow Rate (vR12) Entering Merge Influence Area</th>
<th>Max. Desirable Flow Rate (vR12) Entering Diverge Influence Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥70</td>
<td>4,800 7,200 9,600 2,400/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
<tr>
<td>65</td>
<td>4,700 7,050 9,400 2,350/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
<tr>
<td>60</td>
<td>4,600 6,900 9,200 2,300/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
<tr>
<td>55</td>
<td>4,500 6,750 9,000 2,250/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
</tbody>
</table>

Notes: * Demand in excess of these capacities results in LOS F.  
  a 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)

<table>
<thead>
<tr>
<th>FFS (mi/h)</th>
<th>Capacity of Upstream/Downstream Highway or C-D Segment</th>
<th>Max. Desirable Flow Rate (vR12) Entering Merge Influence Area</th>
<th>Max. Desirable Flow Rate (vR12) Entering Diverge Influence Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥60</td>
<td>4,400 6,600 2,200/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
<tr>
<td>55</td>
<td>4,200 6,300 2,100/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
<tr>
<td>50</td>
<td>4,000 6,000 2,000/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
<tr>
<td>45</td>
<td>3,800 5,700 1,900/ln</td>
<td>4,600</td>
<td>4,400</td>
</tr>
</tbody>
</table>

Notes: * Demand in excess of these capacities results in LOS F.  
  a 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)

<table>
<thead>
<tr>
<th>Ramp FFS SWS (mi/h)</th>
<th>Capacity of Ramp Roadway</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Single-Lane Ramps</td>
</tr>
<tr>
<td>&gt;50</td>
<td>2,200</td>
</tr>
<tr>
<td>&gt;40–50</td>
<td>2,100</td>
</tr>
<tr>
<td>&gt;30–40</td>
<td>2,000</td>
</tr>
<tr>
<td>≥20–30</td>
<td>1,900</td>
</tr>
<tr>
<td>&lt;20</td>
<td>1,800</td>
</tr>
</tbody>
</table>

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 (vFO), or
- Immediately upstream of an off-ramp influence area (vF).

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 (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
Average Speed in

<table>
<thead>
<tr>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ramp influence area</td>
</tr>
<tr>
<td>$D_S = 0.883 + 0.00009v_R - 0.013S_{FS}$</td>
</tr>
<tr>
<td>Outer lanes of freeway</td>
</tr>
<tr>
<td>$S_O = 1.097FFS - 0.0039(v_O - 1,000)$ for $v_O \geq 1,000$ pc/h</td>
</tr>
</tbody>
</table>

### Value

<table>
<thead>
<tr>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average flow in outer lanes $v_{OA}$ (pc/h)</td>
</tr>
<tr>
<td>Average speed for on-ramp (merge) junctions (mi/h)</td>
</tr>
<tr>
<td>Average speed for off-ramp (diverge) junctions (mi/h)</td>
</tr>
</tbody>
</table>

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);
- $S_{FS} =$ 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

\[ D_s = \text{speed index for off-ramps (diverge areas); this is simply an intermediate computation that simplifies the equations.} \]

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.

**Single-Lane Ramp Additions and Lane Drops**

On-ramps and off-ramps do not always include merge and diverge elements. In some cases, there are lane additions at on-ramps or lane drops at off-ramps.

Analysis of single-lane additions and lane drops is relatively straightforward. The freeway segment downstream of the on-ramp or upstream of the off-ramp is simply considered to be a basic freeway segment with an additional lane. The procedures in Chapter 11 should be applied in this case.

The case of an on-ramp lane addition followed by an off-ramp lane drop may be a weaving segment, and should be evaluated using the procedures of Chapter 12. This configuration may either be a weaving segment or a basic segment, depending on the distance between the ramps. Note that some segments may be classified as a weaving segment at higher volumes and as a basic segment at lower volumes.

Ramps with two or more lanes frequently have lane additions or drops for some or all of the ramp lanes. These cases are covered below.
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.

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 \( L_{A0} \), this value is replaced by the effective length of both acceleration lanes \( L_{Aeff} \) from Equation 13-23:

\[
L_{Aeff} = 2L_{A1} + L_{A2}
\]

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 $v_R + (v_L - v_R) \times P_{FD}$, 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: $P_{FD} = 0.260$.

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 $L_{D_{eff}}$ in all equations, obtained from Equation 13-24:

$$L_{D_{eff}} = 2L_{D1} + L_{D2}$$

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.

**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 on-ramps; downstream of off-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

<table>
<thead>
<tr>
<th>Freeway Size</th>
<th>Adjustment Factor for Left-Hand Ramps</th>
</tr>
</thead>
<tbody>
<tr>
<td>On-Ramps</td>
<td>Off-Ramps</td>
</tr>
<tr>
<td>Four-lane</td>
<td>1.00</td>
</tr>
<tr>
<td>Six-lane</td>
<td>1.12</td>
</tr>
<tr>
<td>Eight-lane</td>
<td>1.20</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>On-Ramps</th>
<th>Approaching Freeway Flow</th>
<th>Lane 5 Flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>( v_F ) (pc/h)</td>
<td>( v_5 ) (pc/h)</td>
<td></td>
</tr>
<tr>
<td>( \geq 8,500 )</td>
<td>2.500</td>
<td>( 0.285 \ v_F )</td>
</tr>
<tr>
<td>7,500–8,499</td>
<td>0.285 ( v_F )</td>
<td></td>
</tr>
<tr>
<td>6,500–7,499</td>
<td>0.270 ( v_F )</td>
<td></td>
</tr>
<tr>
<td>5,500–6,499</td>
<td>0.240 ( v_F )</td>
<td></td>
</tr>
<tr>
<td>&lt;5,500</td>
<td>0.220 ( v_F )</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Off-Ramps</th>
<th>Approaching Freeway Flow</th>
<th>Lane 5 Flow</th>
</tr>
</thead>
<tbody>
<tr>
<td>( v_F ) (pc/h)</td>
<td>( v_5 ) (pc/h)</td>
<td></td>
</tr>
<tr>
<td>( \geq 7,000 )</td>
<td>0.200 ( v_F )</td>
<td></td>
</tr>
<tr>
<td>5,500–6,999</td>
<td>0.150 ( v_F )</td>
<td></td>
</tr>
<tr>
<td>4,000–5,499</td>
<td>0.100 ( v_F )</td>
<td></td>
</tr>
<tr>
<td>&lt;4,000</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

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:

\[
v_{F_{4\,eff}} = v_F - v_5
\]

where

\( v_{F_{4\,eff}} \) = effective approaching freeway flow in four lanes (pc/h),

\[\text{Equation 13-25}\]
\[ v_F = \text{total approaching freeway flow in five lanes (pc/h), and} \]
\[ v_5 = \text{estimated approaching freeway flow in Lane 5 (pc/h).} \]

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](image)

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.

![Diagram of major diverge areas](image)

(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:

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

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 11, Basic Freeway Segments, 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.

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**

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**

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**

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

The first criterion that categorizes such applications is the need to use more general estimates of input data. Many of the default values specified in Chapter 11, Basic Freeway Segments; Chapter 12, Freeway Weaving Segments; and Chapter 13, Freeway Merge and Diverge Segments 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**

*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:

\[ SV_i = SF_i \times PHF \]

where

- \( SV_i \) = service volume for LOS i (pc/h),
- \( SF_i \) = service flow rate for LOS i (pc/h), and
- \( PHF \) = peak hour factor.

The method can be applied to determine service volumes for LOS A–E for a specified set of conditions.
There are three categories of general terrain:

- **Level terrain**: Any combination of grades and horizontal or vertical alignment that permits heavy vehicles to maintain the same speed as passenger cars. This type of terrain typically contains short grades of no more than 2%.

- **Rolling terrain**: Any combination of grades and horizontal or vertical alignment that causes heavy vehicles to reduce their speed substantially below that of passenger cars but that does not cause heavy vehicles to operate at crawl speeds for any significant length of time or at frequent intervals. **Crawl speed** is the maximum sustained speed that trucks can maintain on an extended upgrade of a given percent. If the grade is long enough, trucks will be forced to decelerate to the crawl speed, which they can maintain for extended distances. Appendix A of Chapter 11, Basic Freeway Segments, contains truck performance curves that provide truck speeds for various lengths and severities of grade. The same curves may be used for uninterrupted-flow segments on multilane highways.

- **Mountainous terrain**: Any combination of grades and horizontal and vertical alignment that causes heavy vehicles to operate at crawl speed for significant distances or at frequent intervals.

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

Exhibit 14-12 shows PCEs for trucks and buses and RVs in general terrain segments.

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>Level</th>
<th>Rolling</th>
<th>Mountainous</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trucks and buses, $E_T$</td>
<td>1.5</td>
<td>2.5</td>
<td>4.5</td>
</tr>
<tr>
<td>RVs, $E_R$</td>
<td>1.2</td>
<td>2.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

**Equivalents for Specific Upgrades**

Any grade between 2% and 3% and longer than 0.5 mi, or 3% or greater and longer than 0.25 mi, should be considered to be a separate segment. The analysis of such segments must consider the upgrade conditions and the downgrade conditions separately, as well as whether the grade is a single, isolated grade of constant percentage or part of a series forming a composite grade. Appendix A of Chapter 11 discusses the analysis of composite grades.

Exhibit 14-13 and Exhibit 14-14 give values of $E_T$ and $E_R$ for trucks and buses and for RVs, respectively. These factors vary with the percent of grade, length of grade, and the proportion of heavy vehicles in the traffic stream. Maximum values occur when there are only a few heavy vehicles in the traffic stream. The equivalents decrease as the number of heavy vehicles increases because these vehicles tend to form platoons. Because heavy vehicles have more uniform operating characteristics, fewer large gaps are created in the traffic stream when they platoon, and the impact of a single heavy vehicle in a platoon is less severe than that of a single heavy vehicle in a stream primarily composed of passenger...
The grade length should include 25% of the length of the vertical curves at the start and end of the grade. With two consecutive upgrades, 50% of the length of the vertical curve joining them should be included.

The length of the grade is generally taken from a highway profile. It typically includes the straight portion of the grade plus some portion of the vertical curves at the beginning and end of the grade. It is recommended that 25% of the length of the vertical curves at both ends of the grade be included in the length. Where two consecutive upgrades are present, 50% of the length of the vertical curve joining them is included in the length of each grade.
Note that in Exhibit 15-21, the adjustment factor depends on the total two-way demand flow rate, even though the factor is applied to a single directional analysis. The factor reflects not only the percent of no-passing zones in the analysis segment but also the directional distribution of traffic. The directional distribution measure is the same regardless of the direction being considered. Thus, for example, splits of 70/30 and 30/70 result in the same factor, all other variables being constant. Equation 15-9, however, adjusts the factor to reflect the balance of flows in the analysis and opposing directions.

**Step 7: Estimate the PFFS**

This step is included only in the analysis of Class III two-lane highways. PFFS is not used in the determination of LOS for Class I or Class II facilities. The computation is straightforward, since both the FFS and the ATS have already been determined in previous steps. PFFS is estimated from Equation 15-11:

\[
PFFS = \frac{ATS}{FFS}
\]

where all terms are as previously defined.

**Step 8: Determine LOS and Capacity**

**LOS Determination**

At this point in the analysis, the values of any needed measure(s) have been determined. The LOS is found by comparing the appropriate measures with the criteria of Exhibit 15-3. The measure(s) used must be appropriate to the class of the facility being studied:

- Class I: ATS and PTSF;
- Class II: PTSF; and
- Class III: PFFS.

For Class I highways, two service measures are applied. When Exhibit 15-3 is entered, therefore, two LOS designations can be obtained. The worse of the two is the prevailing LOS. For example, if ATS results in a LOS C designation and PTSF results in a LOS D designation, LOS D is assigned.

**Capacity Determination**

Capacity, which exists at the boundary between LOS E and F, is not determined by a measure of effectiveness. Under base conditions, the capacity of a two-lane highway (in one direction) is 1,700 pc/h. To determine the capacity under prevailing conditions, relevant adjustment factors must be applied to Equation 15-3 and Equation 15-7. In this case, however, the demand flow rate of 1,700 pc/h under base conditions is known, and the demand flow rate under prevailing conditions is sought.

First, capacity is defined as a flow rate, so the PHF in Equation 15-3 and Equation 15-7 is set at 1.00. Then, Equation 15-12 or Equation 15-13 (or both) are applied, as described below.
Equation 15-12

\[ c_{ATS} = 1,700 f_{g,ATS} f_{HV,ATS} \]

Equation 15-13

\[ c_{PTSF} = 1,700 f_{g,PTSF} f_{HV,PTSF} \]

where

- \( c_{ATS} \) = capacity in the analysis direction under prevailing conditions based on ATS (veh/h), and
- \( c_{PTSF} \) = capacity in the analysis direction under prevailing conditions based on PTSF (veh/h).

For Class I highways, both capacities must be computed. The lower value represents capacity. For Class II highways, only the PTSF-based capacity is computed. For Class III highways, only the ATS-based capacity is computed.

One complication is that the adjustment factors depend on the demand flow rate (in vehicles per hour). Thus, adjustment factors for a base flow rate of 1,700 pc/h must be used. Technically, this value should be adjusted to reflect grade and heavy vehicle adjustments. This would create an iterative process in which a result is guessed and then checked.

In practical terms, this is unnecessary, since the highest flow group in all adjustment exhibits is greater than 900 veh/h. It is highly unlikely that any adjustments would reduce 1,700 pc/h to less than 900 veh/h. Therefore, in capacity determinations, all adjustment factors should be based on a flow rate greater than 900 veh/h.

Another characteristic of this methodology must be considered in evaluating capacity. When the directional distribution is other than 50/50 (in level and rolling terrain), the two-way capacity implied by each directional capacity may be different. Moreover, the implied two-way capacity from either or both directions may be more than the limit of 3,200 pc/h. In such cases, the directional capacities estimated are not achievable with the stated directional distribution. If this is the case, then base capacity is restricted to 1,700 pc/h in the direction with the heaviest flow, and capacity in the opposing direction is found by using the opposing proportion of flow, with an upper limit of 1,500 pc/h.

**Directional Segments with Passing Lanes**

Providing a passing lane on a two-lane highway in level or rolling terrain improves operational performance and therefore may improve LOS. A procedure to estimate this effect is described in this section.

This procedure should be applied only in level and rolling terrain. On specific grades, added lanes are considered to be climbing lanes, which are addressed in the next section.

Exhibit 15-22 illustrates the operational effect of a passing lane on PTSF. It shows that the passing lane provides operational benefits for some distance downstream before PTSF returns to its former level (without a passing lane). Thus, a passing lane’s effective length is greater than its actual length.
passing lanes or multilane highways), calculate the directional demand flow rate of motorized traffic in the outside lane with Equation 15-24:

\[ v_{OL} = \frac{V}{PHF \times N} \]

where

- \( v_{OL} \) = directional demand flow rate in the outside lane (veh/h),
- \( V \) = hourly directional volume (veh/h),
- \( PHF \) = peak hour factor, and
- \( N \) = number of directional lanes (=1 for two-lane highways).

**Step 3: Calculate the Effective Width**

The effective width of the outside through lane depends on both the actual width of the outside through lane and the shoulder width, since cyclists will be able to travel in the shoulder where one is provided. Moreover, striped shoulders of 4 ft or greater provide more security to cyclists by giving cyclists a dedicated place to ride outside of the motorized vehicle travelway. Thus, an 11-ft lane and adjacent 5-ft paved shoulder results in a larger effective width for cyclists than a 16-ft lane with no adjacent shoulder.

Parking occasionally exists along two-lane highways, particularly in developed areas (Class III highways) and near entrances to recreational areas (Class II and Class III highways) where a fee is charged for off-highway parking or where the off-highway parking is inadequate for the parking demand. On-highway parking reduces the effective width, because parked vehicles take up shoulder space and bicyclists leave some shy distance between themselves and the parked cars.

Equation 15-25 through Equation 15-29 are used to calculate the effective width, \( W_e \), on the basis of the paved shoulder width, \( W_s \), and the hourly directional volume, \( V \):

If \( W_s \) is greater than or equal to 8 ft:

\[ W_e = W_v + W_s - (%OHP \times 10 \text{ ft}) \]

If \( W_s \) is greater than or equal to 4 ft and less than 8 ft:

\[ W_e = W_v + W_s - 2 \times (%OHP(2 \text{ ft} + W_s)) \]

If \( W_s \) is less than 4 ft:

\[ W_e = W_v + (%OHP(2 \text{ ft} + W_s)) \]

with, if \( V \) is greater than 160 veh/h:

\[ W_v = W_{OL} + W_s \]

Otherwise,

\[ W_v = (W_{OL} + W_s) \times (2 - 0.005V) \]

where

- \( W_{OL} \) = effective width as a function of traffic volume (ft).
**Step 4: Calculate the Effective Speed Factor**

The effect of motor vehicle speed on bicycle quality of service is primarily related to the differential between motor vehicle and bicycle travel speeds. For instance, a typical cyclist may travel in the range of 15 mi/h. An increase in motor vehicle speeds from 20 to 25 mi/h is more readily perceived than a speed increase from 60 to 65 mi/h, since the speed differential increases by 100% in the first instance compared with only 11% in the latter. Equation 15-30 shows the calculation of the effective speed factor that accounts for this diminishing effect.

\[
S_t = 1.1199 \ln(S_p - 20) + 0.8103
\]

where

- \(S_t\) = effective speed factor, and
- \(S_p\) = posted speed limit (mi/h).

**Step 5: Determine the LOS**

With the results of Steps 1–4, the bicycle LOS score can be calculated from Equation 15-31:

\[
BLOS = 0.507 \ln(v_{OL}) + 0.1999S_t(1 + 10.38HV)^2 + 7.066(1/P)^2 - 0.005(W_e)^2 + 0.057
\]

where

- \(BLOS\) = bicycle level of service score;
- \(v_{OL}\) = directional demand flow rate in the outside lane (veh/h);
- \(HV\) = percentage of heavy vehicles (decimal); if \(V < 200\) veh/h, then \(HV\) should be limited to a maximum of 50%;
- \(P\) = FHWA’s 5-point pavement surface condition rating; and
- \(W_e\) = average effective width of the outside through lane (ft).

Finally, the BLOS score value is used in Exhibit 15-4 to determine the bicycle LOS for the segment.
It is possible to obtain additional performance measures from simulation results. One example is follower density, which is defined in terms of the number of followers per mile per lane. This concept, which is discussed in more detail in Chapter 24, Concepts: Supplemental, has attracted increasing international interest. Some examples that illustrate potential uses of two-lane highway simulation are presented elsewhere (9).
4. EXAMPLE PROBLEMS

<table>
<thead>
<tr>
<th>Problem Number</th>
<th>Description</th>
<th>Type of Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Find the LOS of a Class I highway in rolling terrain</td>
<td>Operational analysis</td>
</tr>
<tr>
<td>2</td>
<td>Find the LOS of a Class II highway in rolling terrain</td>
<td>Operational analysis</td>
</tr>
<tr>
<td>3</td>
<td>Find the LOS of a Class III highway in level terrain</td>
<td>Operational analysis</td>
</tr>
<tr>
<td>4</td>
<td>Find the LOS of a Class I highway with a passing lane</td>
<td>Operational analysis</td>
</tr>
<tr>
<td>5</td>
<td>Find the future bicycle LOS of a two-lane highway</td>
<td>Planning analysis</td>
</tr>
</tbody>
</table>

EXAMPLE PROBLEM 1: CLASS I HIGHWAY LOS

The Facts

A segment of Class I two-lane highway has the following known characteristics:

- Demand volume = 1,600 veh/h (total in both directions)
- Directional split (during analysis period) = 50/50
- PHF = 0.95
- 50% no-passing zones in the analysis segment (both directions)
- Rolling terrain
- 14% trucks; 4% RVs
- 11-ft lane widths
- 4-ft usable shoulders
- 20 access points/mi
- 60-mi/h BFFS
- 10-mi segment length

Find the expected LOS in each direction on the two-lane highway segment as described.

Comments

The problem statement calls for finding the LOS in each direction on a segment in rolling terrain. Because the directional split is 50/50, the solution in one direction will be the same as the solution in the other direction, so only one operational analysis needs to be conducted. The result will apply equally to each direction.

Because this is a Class I highway, both ATS and PTSF must be estimated to determine the expected LOS.

Step 1: Input Data

All input data were specified above.

Step 2: Estimate the FFS

FFS is estimated with Equation 15-2 and adjustment factors found in Exhibit 15-7 (for lane and shoulder width) and Exhibit 15-8 (for access points in both directions). For 11-ft lane widths and 4-ft usable shoulders, the adjustment factor...
Step 4: Determine Critical Headways and Follow-Up Headways

The critical headways \( t_{c,x} \) and follow-up headways \( t_{f,x} \) must be determined for the major-street left turns \( (v_{c,1} \text{ and } v_{c,4}) \), the minor-street right turns \( (v_{c,9} \text{ and } v_{c,12}) \), the major-street U-turns \( (v_{c,11} \text{ and } v_{c,14}) \), the minor-street through movements \( (v_{c,8} \text{ and } v_{c,13}) \), and the minor-street left turns \( (v_{c,7} \text{ and } v_{c,10}) \) as they occur at a TWSC intersection.

To compute the critical headways for each movement, the analyst begins with the base critical headway given in Exhibit 19-10 and makes movement-specific adjustments relating to the percentage of heavy vehicles, the grade encountered, and a three-leg versus four-leg intersection, as shown in Equation 19-30:

\[
t_{c,x} = t_{c,\text{base}} + t_{c,\text{HV}} P_{HV} + t_{c,G} G - t_{3,LT}
\]

where

- \( t_{c,x} \) = critical headway for movement \( x \) (s);
- \( t_{c,\text{base}} \) = base critical headway from Exhibit 19-10 (s);
- \( t_{c,\text{HV}} \) = adjustment factor for heavy vehicles (1.0 for major streets with one lane in each direction; 2.0 for major streets with two or three lanes in each direction) (s);
- \( P_{HV} \) = proportion of heavy vehicles for movement (expressed as a decimal; e.g., \( P_{HV} = 0.02 \) for 2% heavy vehicles);
- \( t_{c,G} \) = adjustment factor for grade (0.1 for Movements 9 and 12; 0.2 for Movements 7, 8, 10, and 11) (s);
- \( G \) = percent grade (expressed as an integer; e.g., \( G = -2 \) for a 2% downhill grade); and
- \( t_{3,LT} \) = adjustment factor for intersection geometry (0.7 for minor-street left-turn movement at three-leg intersections; 0.0 otherwise) (s).

### Exhibit 19-10

Base Critical Headways for TWSC Intersections

<table>
<thead>
<tr>
<th>Vehicle Movement</th>
<th>Two Lanes</th>
<th>Four Lanes</th>
<th>Six Lanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Left turn from major</td>
<td>4.1</td>
<td>4.1</td>
<td>5.3</td>
</tr>
<tr>
<td>U-turn from major</td>
<td>N/A</td>
<td>6.4 (wide)</td>
<td>5.6</td>
</tr>
<tr>
<td>Right turn from min</td>
<td>6.2</td>
<td>6.9</td>
<td>7.1</td>
</tr>
<tr>
<td>Through traffic on min</td>
<td>1-stage: 6.5</td>
<td>1-stage: 6.5</td>
<td>1-stage: 6.5*</td>
</tr>
<tr>
<td></td>
<td>2-stage, Stage I: 5.5</td>
<td>2-stage, Stage I: 5.5</td>
<td>2-stage, Stage I: 5.5*</td>
</tr>
<tr>
<td></td>
<td>2-stage, Stage II: 5.5</td>
<td>2-stage, Stage II: 5.5</td>
<td>2-stage, Stage II: 5.5*</td>
</tr>
<tr>
<td>Left turn from min</td>
<td>1-stage: 7.1</td>
<td>1-stage: 7.5</td>
<td>1-stage: 6.4</td>
</tr>
<tr>
<td></td>
<td>2-stage, Stage I: 6.1</td>
<td>2-stage, Stage I: 6.5</td>
<td>2-stage, Stage I: 7.3</td>
</tr>
<tr>
<td></td>
<td>2-stage, Stage II: 6.1</td>
<td>2-stage, Stage II: 6.5</td>
<td>2-stage, Stage II: 6.7</td>
</tr>
</tbody>
</table>

* Use caution; values estimated.

Note: "Narrow" U-turns have a median nose width < 21 ft; "wide" U-turns have a median nose width ≥21 ft.

The critical headway data for four- and six-lane sites account for the actual lane distribution of traffic flows measured at each site. For six-lane sites, minor-street left turns were commonly observed beginning their movement while apparently conflicting vehicles in the far-side major-street through stream pass. The values for critical headway for minor-street through movements at six-lane streets are estimated, as the movement is not frequently observed in the field.
Similar to the computation of critical headways, the analyst begins the computation of follow-up headways with the base follow-up headways given in Exhibit 19-11. The analyst then makes movement-specific adjustments to the base follow-up headways with information gathered on heavy vehicles and the geometrics of the major street per the adjustment factors given in Equation 19-31.

\[ t_{f,x} = t_{f,\text{base}} + t_{f,HV} P_{HV} \]

where

- \( t_{f,x} \) = follow-up headway for movement \( x \) (s),
- \( t_{f,\text{base}} \) = base follow-up headway from Exhibit 19-11 (s),
- \( t_{f,HV} \) = adjustment factor for heavy vehicles (0.9 for major streets with one lane in each direction, 1.0 for major streets with two or three lanes in each direction), and
- \( P_{HV} \) = proportion of heavy vehicles for movement (expressed as a decimal; e.g., \( P_{HV} = 0.02 \) for 2% heavy vehicles).

### Base Follow-Up Headways for TWSC Intersections

<table>
<thead>
<tr>
<th>Vehicle Movement</th>
<th>Base Follow-Up Headway, ( t_{f,\text{base}} ) (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Two Lanes</td>
</tr>
<tr>
<td>Left turn from major</td>
<td>2.2</td>
</tr>
<tr>
<td>U-turn from major</td>
<td>N/A</td>
</tr>
<tr>
<td>Right turn from minor</td>
<td>3.3</td>
</tr>
<tr>
<td>Through traffic on minor</td>
<td>4.0</td>
</tr>
<tr>
<td>Left turn from minor</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Note: “Narrow” U-turns have a median nose width < 21 ft; “wide” U-turns have a median nose width ≥21 ft.

Values from Exhibit 19-10 and Exhibit 19-11 are based on studies throughout the United States and are representative of a broad range of conditions. If smaller values for \( t_c \) and \( t_f \) are observed, capacity will be increased. If larger values for \( t_c \) and \( t_f \) are used, capacity will be decreased.

### Step 5: Compute Potential Capacities

#### Step 5a: Potential Capacity If No Upstream Signal Effects Are Present

The potential capacity \( c_{p,x} \) of a movement is computed according to the gap-acceptance model provided in Equation 19-32 (6). This model requires the analyst to input the conflicting flow rate \( v_{c,x} \), the critical headway \( t_{c,x} \), and the follow-up headway \( t_{f,x} \) for movement \( x \).

\[ c_{p,x} = v_{c,x} \frac{e^{-v_{c,x} t_{c,x} / 3,600}}{1 - e^{-v_{c,x} t_{f,x} / 3,600}} \]

where

- \( c_{p,x} \) = potential capacity of movement \( x \) (veh/h),
- \( v_{c,x} \) = conflicting flow rate for movement \( x \) (veh/h),
- \( t_{c,x} \) = critical headway for minor movement \( x \) (s), and
- \( t_{f,x} \) = follow-up headway for minor movement \( x \) (s).
### Step 10: Compute Departure Headways

The departure headway of the lane is the expected value of the saturation headway distribution, given by Equation 20-28.

$$h_d = \sum_{i=1}^{64} P(i)h_{si}$$

where $i$ represents each combination of the five degree-of-conflict cases and $h_{si}$ is the saturation headway for that combination.

### Step 11: Check for Convergence

The calculated values of $h_d$ are checked against the initial values assumed for $h_d$. If the values change by more than 0.1 s (or a more precise measure of convergence), Steps 5 through 10 are repeated until the values of departure headway for each lane do not change significantly.

### Step 12: Compute Capacity

The capacity of each lane in a subject approach is computed under the assumption that the flows on the opposing and conflicting approaches are constant. The given flow rate on the subject lane is increased and the departure headways are computed for each lane on each approach until the degree of utilization for the subject lane reaches 1. When this occurs, the final value of the subject lane flow rate is the maximum possible throughput or capacity of this lane.

### Step 13: Compute Service Times

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time with Equation 20-29.

$$t_s = h_d - m$$

where $t_s$ is the service time, $h_d$ is the departure headway, and $m$ is the move-up time (2.0 s for Geometry Groups 1 through 4; 2.3 s for Geometry Groups 5 and 6).
Step 14: Compute Control Delay for Each Lane

The delay experienced by a motorist is made up of a number of factors that relate to control, geometrics, traffic, and incidents. Control delay is the difference between the travel time that is actually experienced and the reference travel time that would result during conditions in the absence of traffic control or conflicting traffic.

Equation 20-30 can be used to compute control delay for each lane.

\[
   d = t_s + 900T \left[ (x-1) + \sqrt{(x-1)^2 + \frac{h_d x}{450T}} + 5 \right]
\]

where

- \( d \) = average control delay (s/veh),
- \( x = \frac{vh_d}{3600} \) = degree of utilization,
- \( t_s \) = service time (s),
- \( h_d \) = departure headway (s), and
- \( T \) = length of analysis period (h).

Step 15: Compute Control Delay and Determine LOS for Each Approach and the Intersection

The control delay for an approach is calculated by computing a weighted average of the delay for each lane on the approach, weighted by the volume in each lane. The calculation is shown in Equation 20-31.

\[
   d_{\text{approach}} = \frac{\sum d_i v_i}{\sum v_i}
\]

where

- \( d_{\text{approach}} \) = control delay for the approach (s/veh),
- \( d_i \) = control delay for lane \( i \) (s/veh), and
- \( v_i \) = flow rate for lane \( i \) (veh/h).

The control delay for the intersection as a whole is similarly calculated by computing a weighted average of the delay for each approach, weighted by the volume on each approach. It is shown in Equation 20-32.

\[
   d_{\text{intersection}} = \frac{\sum d_i v_i}{\sum v_i}
\]

where

- \( d_{\text{intersection}} \) = control delay for the entire intersection (s/veh),
- \( d_i \) = control delay for approach \( i \) (s/veh), and
- \( v_i \) = flow rate for approach \( i \) (veh/h).

The LOS for each approach and for the intersection are determined with Exhibit 20-2 and the computed values of control delay.
\[ AdjP(5) = 0.01[0.052 + 2(0) - 3(0.088)]/6 = -0.0004 \]
\[ AdjP(16) = 0.01[0 - 6(0.052)]/27 = -0.0001 \]

Therefore, the adjusted probability for Combination 1, for example, is as follows:

\[ P'(1) = 0.538 + 0.0065 = 0.5445 \]

**Step 9: Compute Saturation Headways**

The base saturation headways for each combination can be determined with Exhibit 20-14. They are adjusted by using the adjustment factors calculated in Step 4 and added to the base saturation headways to determine saturation headways as follows (eastbound illustrated):

<table>
<thead>
<tr>
<th>( i )</th>
<th>( h_{base} )</th>
<th>( h_{adj} )</th>
<th>( h_{adj} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3.9</td>
<td>0.063</td>
<td>3.963</td>
</tr>
<tr>
<td>2</td>
<td>4.7</td>
<td>0.063</td>
<td>4.763</td>
</tr>
<tr>
<td>5</td>
<td>5.8</td>
<td>0.063</td>
<td>5.863</td>
</tr>
<tr>
<td>7</td>
<td>7.0</td>
<td>0.063</td>
<td>7.063</td>
</tr>
</tbody>
</table>

**Step 10: Compute Departure Headways**

The departure headway of the lane is the sum of the products of the adjusted probabilities and the saturation headways as follows (eastbound illustrated):

\[ h_d = (0.5445)(3.963) + (0.3213)(4.763) + (0.0875)(5.863) + (0.0524)(7.063) = 4.57 \]

**Step 11: Check for Convergence**

The calculated values of \( h_d \) are checked against the initial values assumed for \( h_d \). After one iteration, each calculated headway differs from the initial value by more than 0.1 s. Therefore, the new calculated headway values are used as initial values in a second iteration. For this problem, four iterations are required for convergence.

<table>
<thead>
<tr>
<th></th>
<th>EB L1</th>
<th>EB L2</th>
<th>WB L1</th>
<th>WB L2</th>
<th>NB L1</th>
<th>NB L2</th>
<th>SB L1</th>
<th>SB L2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Lane Flow Rate</td>
<td>368</td>
<td>421</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( h_d ), initial value, iteration 1</td>
<td>3.2</td>
<td>3.2</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( x ), initial, iteration 1</td>
<td>0.327</td>
<td>0.374</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( h_d ), computed value, iteration 1</td>
<td>4.57</td>
<td>4.35</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Convergence?</td>
<td>N</td>
<td>N</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( h_d ), initial value, iteration 2</td>
<td>4.57</td>
<td>4.35</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( x ), initial, iteration 2</td>
<td>0.468</td>
<td>0.509</td>
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<tr>
<td>( h_d ), computed value, iteration 2</td>
<td>4.88</td>
<td>4.66</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Convergence?</td>
<td>N</td>
<td>N</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( h_d ), initial value, iteration 3</td>
<td>4.88</td>
<td>4.66</td>
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<td></td>
</tr>
<tr>
<td>( x ), initial, iteration 3</td>
<td>0.499</td>
<td>0.545</td>
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<td></td>
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</tr>
<tr>
<td>( h_d ), computed value, iteration 3</td>
<td>4.95</td>
<td>4.73</td>
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<td></td>
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</tr>
<tr>
<td>Convergence?</td>
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<td>Y</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( h_d ), initial value, iteration 4</td>
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<td>4.66</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( x ), initial, iteration 4</td>
<td>0.499</td>
<td>0.545</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( h_d ), computed value, iteration 4</td>
<td>4.97</td>
<td>4.74</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Convergence?</td>
<td>Y</td>
<td>Y</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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July 2012
**Step 12: Compute Capacity**

The capacity of each lane in a subject approach is computed by increasing the given flow rate on the subject lane (assuming the flows on the opposing and conflicting approaches are constant) until the degree of utilization for the subject lane reaches 1. This level of calculation requires running an iterative procedure many times, which is practical for a spreadsheet or software implementation.

Here, the eastbound lane capacity is approximately 720 veh/h, which is lower than the value that could be estimated by dividing the lane volume by the degree of utilization (368/0.492 = 748 veh/h). The difference is due to the interaction effects among the approaches: increases in eastbound traffic volume increase the departure headways of the lanes on the other approaches, which in turn increases the departure headway of the lane(s) on the subject approach.

**Step 13: Compute Service Times**

The service time required to calculate control delay is computed on the basis of the final calculated departure headway and the move-up time by using Equation 20-29. For the eastbound lane (using a value for \( m \) of 2.0 for Geometry Group 1), the calculation is as follows:

\[
t_s = h_d - m = 4.97 - 2.0 = 2.97
\]

**Step 14: Compute Control Delay**

The control delay for each lane is computed with Equation 20-30 as follows (eastbound illustrated):

\[
d = 2.97 + 900(0.25) \left[ (0.508 - 1) + \sqrt{(0.508 - 1)^2 + \frac{4.97(0.508)}{450(0.25)}} \right] + 5 = 13.0 \text{ s}
\]

By using Exhibit 20-2, the eastbound lane (and thus approach) is assigned LOS B. A similar calculation for the westbound and southbound lanes (and thus approaches) yields 13.5 and 10.6 s, respectively.

The control delays for the approaches can be combined into an intersection control delay by using a weighted average as follows:

\[
d = \frac{(13.0)(368) + (13.5)(421) + (10.6)(158)}{368 + 421 + 158} = 12.8 \text{ s}
\]

This value of delay is assigned LOS B.

**Step 15: Compute Queue Length**

The 95th percentile queue for each lane is computed with Equation 20-33 as follows (eastbound approach illustrated):

\[
Q_{95} \approx \frac{900(0.25)}{4.97} \left[ (0.508 - 1) + \sqrt{(0.508 - 1)^2 + \frac{4.97(0.508)}{150(0.25)}} \right] = 2.9 \text{ veh}
\]

This queue length would be reported as 3 vehicles.