CHAPTER 28
FREEWAY MERGES AND DIVERGES: SUPPLEMENTAL

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1. INTRODUCTION

Chapter 28 is the supplemental chapter for Chapter 14, Freeway Merge and Diverge Segments, which is found in Volume 2 of the *Highway Capacity Manual* (HCM). Section 2 provides five example problems demonstrating the application of the Chapter 14 methodology and its extension to freeway managed lanes. Section 3 presents examples of applying alternative tools to the analysis of freeway merge and diverge segments to address limitations of the Chapter 14 methodology.
2. EXAMPLE PROBLEMS

Exhibit 28-1 lists the example problems presented in this section.

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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. The example assumes no impacts of inclement weather or incidents.

1. Isolated location (no adjacent ramps to consider);
2. One-lane ramp roadway and junction;
3. Four-lane freeway (two lanes in each direction);
4. Upstream freeway demand volume = 2,500 veh/h;
5. Ramp demand volume = 535 veh/h;
6. 5% trucks throughout;
7. Acceleration lane = 740 ft;
8. FFS, freeway = 60 mi/h;
9. FFS, ramp = 45 mi/h;
10. Level terrain for freeway and ramp;
11. Peak hour factor (PHF) = 0.90; and
12. 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 12, Basic Freeway and Multilane Highway Segments.
Step 1: Specify Inputs and Convert Demand Volumes to Demand Flow Rates

Input parameters were specified in the Facts section above. Equation 14-1 is used to convert demand volumes to flow rates under equivalent ideal conditions:

\[ v_i = \frac{V_i}{PHF \times f_{hv}} \]

Demand volumes are given for the freeway and the ramp. The PHF is specified. The driver population adjustment factors for commuters are 1.00 (Chapter 12), while the heavy vehicle adjustment factor is computed as follows:

\[ f_{hv} = \frac{1}{1 + P_T(E_T - 1)} \]

Truck presence is given. The value of \( E_T \) for level terrain is 2.0 (Chapter 12). On the basis of these values, the freeway and ramp demand volumes are converted as follows:

For the freeway,

\[ f_{hv} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.05(2.0 - 1)} = 0.952 \]

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

For the ramp, the calculations are identical:

\[ f_{hv} = \frac{1}{1 + 0.05(2.0 - 1)} = 0.952 \]

\[ v_R = \frac{535}{0.90 \times 0.952} = 625 \text{ pc/h} \]

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

The demand flow in Lanes 1 and 2 immediately upstream of the ramp influence area is computed by using Equation 14-2.

\[ v_{12} = v_F \times P_{FM} \]

The freeway flow rate was computed in Step 1. The value of \( P_{FM} \) is found in Exhibit 14-8. 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: Estimate the Capacity of the Ramp–Freeway Junction and Compare with Demand Flow Rates

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

\[ v_{FO} = v_F + v_R = 2,918 + 625 = 3,543 \text{ pc/h} \]
The capacity of a four-lane freeway (two lanes in one direction) with an FFS of 60 mi/h is given in Exhibit 14-10. The capacity is 4,600 pc/h, which is more than the demand flow of 3,543 pc/h. The capacity of a one-lane ramp with an FFS of 45 mi/h is given in Exhibit 14-12 as 2,100 pc/h, which is well in excess of the ramp demand flow of 625 pc/h. The maximum desirable flow rate entering the ramp influence area is also 4,600 pc/h, again more than 3,543. Thus, the operation of the segment is expected to be stable. LOS F does not exist. Note that there were no adjustments to speed (SAF) or capacity (CAF) due to inclement weather, incidents, or other impacts for this case.

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

The estimated density in the ramp–freeway junction is estimated by using Equation 14-22:

\[ D_R = 5.475 + 0.00734v_R + 0.0078v_{12} - 0.00627L_A \]

\[ D_R = 5.475 + 0.00734(625) + 0.0078(2,918) - 0.00627(740) \]

\[ D_R = 28.2 \text{ pc/mi} \ln \]

From Exhibit 14-3, this is LOS D, but the result is close to the LOS C boundary.

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

Since there are no outer lanes on a four-lane freeway, only the speed within the ramp influence area should be computed, by using the equations given in Exhibit 14-13:

\[ M_S = 0.321 + 0.0039e^{(v_{R12}/1,000)} - 0.002(L_A \times S_{FR} \times SAF/1,000) \]

\[ M_S = 0.321 + 0.0039(3.543/1,000) - 0.002(740 \times 45 \times 100/1,000) = 0.389 \]

\[ S_R = FFS \times SAF - (FFS \times SAF - 42)M_S \]

\[ S_R = 60 \times 1.00 - (60 \times 1.00 - 42)(0.389) = 53.0 \text{ mi/h} \]

Note that the speed adjustment factor, SAF, is 1.00, since this is not a case where inclement weather or other factors would necessitate a correction.

**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. The example assumes no impacts of inclement weather or incidents.

1. Two consecutive one-lane, right-hand off-ramps;
2. Six-lane freeway with FFS = 60 mi/h;
3. Level terrain for freeway and both ramps;
4. 7.5% trucks on freeway and both ramps;
5. First-ramp FFS = 40 mi/h;
6. Second-ramp FFS = 25 mi/h;
7. Drivers are regular commuters;
8. Freeway demand volume = 4,500 veh/h (immediately upstream of the first off-ramp);
9. First-ramp demand volume = 300 veh/h;
10. Second-ramp demand volume = 500 veh/h;
11. Distance between ramps = 750 ft;
12. First-ramp deceleration lane length = 500 ft;
13. Second-ramp deceleration lane length = 300 ft; and
14. Peak hour factor = 0.95.

Comments
The solution will use adjustment factors for heavy vehicle presence and driver population selected from Chapter 12, Basic Freeway and Multilane Highway Segments. All input parameters are specified, so no default values are needed or used.

Step 1: Specify Inputs and Convert Demand Volumes to Demand Flow Rates
Input parameters were specified in the Facts section above. Equation 14-1 is used to convert demand volumes to flow rates under equivalent ideal conditions:

\[ v_i = \frac{V_i}{PHF \times f_{HV}} \]

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 7.5% trucks, only a single heavy vehicle adjustment factor will be needed. From Chapter 12, the appropriate value of \( E_T \) for level terrain is 2.0.

Then

\[ f_{HV} = \frac{1}{1 + P_T (E_T - 1)} = \frac{1}{1 + 0.075(2 - 1)} = 0.930 \]

and

\[ v_F = \frac{4,500}{0.95 \times 0.930} = 5,093 \text{ pc/h} \]
\[ v_{R1} = \frac{300}{0.95 \times 0.930} = 340 \text{ pc/h} \]
\[ v_{R2} = \frac{500}{0.95 \times 0.930} = 566 \text{ pc/h} \]
Step 2: Estimate the Approaching Flow Rate in Lanes 1 and 2 of the Freeway Immediately Upstream of the Ramp Influence Area

Because two consecutive off-ramps are 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 14-9, flow in Lanes 1 and 2 of the freeway is estimated by using Equation 14-11 or Equation 14-9, depending on whether the impact of the downstream off-ramp is significant. This is determined by computing the equivalence distance by using Equation 14-13:

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

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

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 14-9:

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

\[
P_{FD} = 0.760 - 0.000025(5,093) - 0.000046(340) = 0.617
\]

Then from Equation 14-8,

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

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

Because a six-lane freeway includes one lane in addition to the ramp influence areas (the innermost 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:

\[
\text{Is } v_3 > 2,700 \text{ pc/h/ln?} \quad \text{No}
\]

\[
\text{Is } v_3 > 1.5 \times (1,637) = 2,456 \text{ pc/h/ln?} \quad \text{No}
\]

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 14-9, the second off-ramp should be analyzed by using Equation 14-9, 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

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

\[ P_{FD} = 0.760 - 0.000025(4753) - 0.000046(566) = 0.615 \]

\[ v_{12} = 566 + (4753 - 566)(0.615) = 3141 \text{ pc/h} \]

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 innermost lane \( v_3 \) is 4753 – 3141 = 1612 pc/h. The average flow rate in Lanes 1 and 2 is 3141/2 = 1571 pc/h (rounded). Then:

\[ \text{Is } v_3 > 2700 \text{ pc/h/ln?} \quad \text{No} \]

\[ \text{Is } v_3 > 1.5 \times 1571 = 2357 \text{ pc/h/ln?} \quad \text{No} \]

Once again, the predicted lane distribution of arriving vehicles is reasonable, and \( v_{12} \) is taken to be 3141 pc/h.

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

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

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

These comparisons are shown in Exhibit 28-2. 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.

<table>
<thead>
<tr>
<th>Item</th>
<th>Capacity (pc/h) from Exhibit 14-10 or Exhibit 14-12</th>
<th>Demand Flow Rate (pc/h)</th>
<th>Problem?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway flow rate</td>
<td>6900</td>
<td>5093</td>
<td>No</td>
</tr>
<tr>
<td>First off-ramp</td>
<td>2000</td>
<td>340</td>
<td>No</td>
</tr>
<tr>
<td>Second off-ramp</td>
<td>1900</td>
<td>566</td>
<td>No</td>
</tr>
<tr>
<td>Max. ( v_{12} ) first ramp</td>
<td>4400</td>
<td>3273</td>
<td>No</td>
</tr>
<tr>
<td>Max. ( v_{12} ) second ramp</td>
<td>4400</td>
<td>3141</td>
<td>No</td>
</tr>
</tbody>
</table>

Note: \( \text{Max.} = \text{maximum}. \)

None of the capacity values are exceeded, so operation of these ramp junctions will be stable, and LOS F does not occur. Again, there are no situations that would call for an adjustment to be made to speed (SAF) or capacity (CAF).

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

Because there are two off-ramps, two ramp influence areas are involved, and two ramp influence area densities will be computed with Equation 14-23.

\[ D_R = 4.252 + 0.0086v_{12} - 0.009L_D \]

\[ D_{R1} = 4.252 + 0.0086(3273) - 0.009(500) = 27.9 \text{ pc/mi/ln} \]

\[ D_{R2} = 4.252 + 0.0086(3141) - 0.009(300) = 28.6 \text{ pc/mi/ln} \]
From Exhibit 14-3, both of these ramp influence areas operate 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.

Although 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. Since 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). The worse of the two levels of service is applied to this 750-ft overlap. In this case, the levels of service are different, even though the predicted densities are similar. Thus, the overlapping influence area is assigned LOS D.

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

Because these ramps are on a six-lane freeway with an outer lane, 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 can be estimated.

**First Off-Ramp**

The speed within the first ramp influence area is computed by using the equations given in Exhibit 14-14:

\[ D_s = 0.883 + 0.00009v_r - 0.013S_{FR} \times SAF \]
\[ D_s = 0.883 + 0.00009(340) - 0.013(40)(1.00) = 0.394 \]
\[ S_R = FFS \times SAF - (FFS \times SAF - 42)D_s \]
\[ S_R = (60)(1.00) - (60 \times 1.00 - 42)(0.394) = 52.9 \text{ mi/h} \]

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, by using the equation given in Exhibit 14-14:

\[ v_v = 3.273 \times (1.00) - 0.0039(1,820 - 1,000) = 62.6 \text{ mi/h} \]

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. However, the average speed across all lanes should not be higher than the FFS. In this case, the average speed across all lanes is computed as follows, by using the appropriate equation from Exhibit 14-15:

\[ S = \frac{v_{12} + v_{OA}N_O}{(S_R) + (\frac{v_{OA}N_O}{S_D})} = \frac{3,273 + (1,820)(1)}{(3,273 \times 52.9) + (1,820 \times 1 \times 62.6)} = 56.0 \text{ mi/h} \]

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

Note that once again the SAF is 1.00, since there are no conditions that would require an adjustment.
Second Off-Ramp

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

\[ D_S = 0.883 + 0.00009(566) - 0.013(25)(1.00) = 0.609 \]
\[ S_R = (60)(1.00) - (60 \times 1.00 - 42)(0.609) = 49.0 \text{ mi/h} \]

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:

\[ S_0 = (1.097)(60)(1.00) - 0.0039(1,612 - 1,000) = 63.4 \text{ mi/h} \]

The average speed across all freeway lanes is

\[ S = \frac{v_{12} + v_{OA}N_O}{S_R} = \frac{3,141 + (1,612)(1)}{(3,141 \div 49.0) + \frac{1,612 \times 1}{63.4}} = 53.1 \text{ mi/h} \]

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. In this case the step-function LOS assigned may imply 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. The example assumes no impacts of inclement weather or incidents.

1. Eight-lane freeway with an FFS of 65 mi/h;
2. One-lane, right-hand on-ramp with an FFS of 30 mi/h;
3. One-lane, right-hand off-ramp with an FFS of 25 mi/h;
4. Distance between ramps = 1,300 ft;
5. Acceleration lane on Ramp 1 = 260 ft;
6. Deceleration lane on Ramp 2 = 260 ft;
7. Level terrain on freeway and both ramps;
8. 10% trucks on freeway and off-ramp;
9. 5% trucks on on-ramp;
10. Freeway flow rate (upstream of first ramp) = 5,490 veh/h;
11. On-ramp flow rate = 410 veh/h;
12. Off-ramp flow rate = 600 veh/h;
13. PHF = 0.94; and
14. Drivers are regular commuters.

Comments
As with previous example problems, the conversion of demand volumes to flow rates requires adjustment factors selected from Chapter 12, Basic Freeway and Multilane Highway Segments. All pertinent information is given, and no default values will be applied.

Step 1: Specify Inputs and Convert Demand Volumes to Demand Flow Rates
Input parameters were specified in the Facts section above. Equation 14-1 is used to convert demand volumes to flow rates under equivalent ideal conditions:

\[ v_i = \frac{V_i}{\text{PHF} \times f_{HV}} \]

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 2.0 for all volumes.

Then, for the freeway demand volume,

\[ f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.10(2 - 1)} = 0.91 \]

\[ v_F = \frac{5490}{0.94 \times 0.91} = 6418 \text{ pc/h} \]

For the on-ramp demand volume,

\[ f_{HV} = \frac{1}{1 + 0.05(2 - 1)} = 0.952 \]

\[ v_{R1} = \frac{410}{0.94 \times 0.952} = 458 \text{ pc/h} \]

For the off-ramp demand volume,

\[ f_{HV} = \frac{1}{1 + 0.10(2 - 1)} = 0.91 \]

\[ v_{R2} = \frac{600}{0.94 \times 0.91} = 701 \text{ pc/h} \]

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

Step 2: Estimate the Approaching Flow Rate in Lanes 1 and 2 of the Freeway Immediately Upstream of the Ramp Influence Area
Once again, the situation involves a pair of adjacent ramps. Analysis of each ramp must take into account the potential impact of the other on its operations. Because the ramps are on an eight-lane freeway (four lanes in each direction), Exhibit 14-8 and Exhibit 14-9 indicate that each ramp is considered as if it were isolated.
**First Ramp: On-Ramp**

Exhibit 14-8 applies to on-ramps. Exhibit 14-8 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 $6,418/30 = 213.9 > 72$. Therefore, the second equation for eight-lane freeways given in Exhibit 14-8 is used, giving the following:

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

$$P_{FM} = 0.2178 - 0.000125v_{R} = 0.2178 - 0.000125(458) = 0.16$$

$$v_{12} = (6,418)(0.16) = 1,027 \text{ pc/h}$$

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 $1,027/2 = 514 \text{ pc/h/ln}$ (rounded). The flow in the two outer lanes, Lanes 3 and 4, is $6,418 - 1,027 = 5,391 \text{ pc/h}$. The average flow per lane in Lanes 3 and 4 is, therefore, $5,391/2 = 2,696 \text{ pc/h/ln}$. Then:

Is $v_{av,34} > 2,700 \text{ pc/h/ln}$? No

Is $v_{av,34} > 1.5 \times 514 = 771 \text{ pc/h/ln}$? Yes

Therefore, the predicted lane distribution is not reasonable. Too many vehicles are placed in the two outer lanes compared with Lanes 1 and 2. Equation 14-19 is used to produce a more reasonable distribution:

$$v_{12a} = \left(\frac{v_{F}}{2.50}\right) = \left(\frac{6,418}{2.50}\right) = 2,567 \text{ pc/h}$$

On the basis of this adjusted value, the number of vehicles now assigned to the two outer lanes is $6,418 - 2,567 = 3,851 \text{ pc/h}$.

**Second Ramp: Off-Ramp**

Equation 14-8 and Exhibit 14-9 apply to off-ramps. Exhibit 14-9 shows that the value of $P_{FD}$ for off-ramps on eight-lane freeways is a constant: 0.436. Since 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 $6,418 + 458 = 6,876 \text{ pc/h}$. The flow rate in Lanes 1 and 2 is now easily computed by using Equation 14-8:

$$v_{12} = v_{R} + (v_{F} - v_{R})P_{FD}$$

$$v_{12} = 701 + (6,876 - 701)(0.436) = 3,393 \text{ pc/h}$$

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 $3,393/2 = 1,697 \text{ pc/h/ln}$. The total flow in Lanes 3 and 4 of the freeway is $6,876 - 3,393 = 3,483 \text{ pc/h}$, or an average flow rate per lane of $3,483/2 = 1,742 \text{ pc/h/ln}$.

Is $v_{av,34} > 2,700 \text{ pc/h/ln}$? No

Is $v_{av,34} > 1.5 \times 1,697 = 2,545 \text{ pc/h/ln}$? No

Therefore, the estimated value of $v_{12}$ is deemed reasonable and is carried forward in the computations.
**Step 3: Estimate the Capacity of the Ramp–Freeway Junction and Compare with Demand Flow Rates**

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

1. 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.
2. The capacity of the on-ramp.
3. The capacity of the off-ramp.
4. The maximum desirable flow entering the on-ramp influence area.
5. The maximum desirable flow entering the off-ramp influence area.

These comparisons are shown in Exhibit 28-3. 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.

<table>
<thead>
<tr>
<th>Item</th>
<th>Capacity (pc/h) from Exhibit 14-10 or Exhibit 14-12</th>
<th>Demand Flow Rate (pc/h)</th>
<th>Problem?</th>
</tr>
</thead>
<tbody>
<tr>
<td>Freeway flow rate</td>
<td>9,400</td>
<td>6,876</td>
<td>No</td>
</tr>
<tr>
<td>First on-ramp</td>
<td>1,900</td>
<td>458</td>
<td>No</td>
</tr>
<tr>
<td>Second off-ramp</td>
<td>1,900</td>
<td>701</td>
<td>No</td>
</tr>
<tr>
<td>Max. $v_{R_1}$, first ramp</td>
<td>4,600</td>
<td>2,567 + 458 = 3,025</td>
<td>No</td>
</tr>
<tr>
<td>Max. $v_{R_2}$, second ramp</td>
<td>4,400</td>
<td>3,393</td>
<td>No</td>
</tr>
</tbody>
</table>

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

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

Equation 14-22 is used to find the density in the first on-ramp influence area:

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

$$D_R = 5.475 + 0.00734(458) + 0.0078(2,567) - 0.00627(260)$$

$$D_R = 27.2 \text{ pc/mi/ln}$$

Equation 14-23 is used to find the density in the second off-ramp influence area:

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

$$D_R = 4.252 + 0.0086(3,393) - 0.009(260) = 31.1 \text{ pc/mi/ln}$$

From Exhibit 14-3, both of these ramp influence areas operate 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 lower LOS D for the off-ramp governs the operation. Note that 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: Estimate Speeds in the Vicinity of Ramp–Freeway Junctions**

Because the facility is an 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 areas, 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 14-13.

\[ M_S = 0.321 + 0.0039e^{(v_{R12}/1,000)} - 0.002(L_A \times S_{FR} \times SAF/1,000) \]
\[ M_S = 0.321 + 0.0039e^{(3.025/1,000)} - 0.002(260 \times 30 \times 1.00/1,000) = 0.385 \]
\[ S_R = FFS \times SAF - (FFS \times SAF - 42)M_S \]
\[ S_R = (65)(1.00) - (65 \times 1.00 - 42)(0.385) = 56.2 \text{ mi/h} \]

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, by using the appropriate equation from Exhibit 14-13:

\[ S_O = FFS \times SAF - 0.0036(v_{OA} - 500) \]
\[ S_O = (65)(1.00) - 0.0036(1,926 - 500) = 59.9 \text{ mi/h} \]

Note that the speed adjustment factor (SAF) is 1.00.

The weighted average speed of all vehicles is

\[ S = \frac{v_{12} + v_{OA}N_O}{S_R} + \frac{v_{OA}N_O}{S_O} = \frac{3,025 + (1,926)(2)}{56.2 + \frac{(1,926 \times 2)}{59.9}} = 58.2 \text{ mi/h} \]

**Second Ramp (Off-Ramp)**

For off-ramps, equations for estimation of average speed are drawn from Exhibit 14-14. 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 3,483 pc/h or 1,742 pc/h/ln. Then

\[ D_S = 0.883 + 0.00009v_R - 0.013S_{FR} \times SAF \]
\[ D_S = 0.883 + 0.00009(701) - 0.013(25)(1.00) = 0.621 \]
\[ S_R = FFS \times SAF - (FFS \times SAF - 42)D_S \]
\[ S_R = (65)(1.00) - (65 \times 1.00 - 42)(0.621) = 50.7 \text{ mi/h} \]

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:

\[ S_O = 1.097 \times FFS \times SAF - 0.0039(v_{OA} - 1,000) \]
\[ S_O = (1.097)(65)(1.00) - 0.0039(1,742 - 1,000) = 68.4 \text{ mi/h} \]
The weighted average speed of all vehicles is

\[
S = \frac{v_{12} + v_{04}N_D}{\left(\frac{v_{12}}{S_R}\right) + \left(\frac{V_{04}N_D}{S_O}\right)} = \frac{3,393 + (1,742)(2)}{\left(\frac{3,393}{50.7}\right) + \left(\frac{1,742 \times 2}{68.4}\right)} = 58.3 \text{ mi/h}
\]

**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 for those 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, in contrast, can execute as many lane changes as they wish, 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**

The following information is available concerning this example problem. The example assumes no impacts of inclement weather or incidents.

1. One-lane, left-side on-ramp on a six-lane freeway (three lanes in each direction);
2. Freeway demand volume upstream of ramp = 4,000 veh/h;
3. On-ramp demand volume = 490 veh/h;
4. 7.5% trucks on freeway, 3% trucks on the on-ramp;
5. Freeway FFS = 65 mi/h;
6. Ramp FFS = 30 mi/h;
7. Acceleration lane = 820 ft;
8. Level terrain on freeway and ramp;
9. Drivers are regular commuters; and
10. PHF = 0.90.
Comments

This is a special application of the ramp analysis methodology presented in Chapter 14. 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 14-18 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: Specify Inputs and Convert Demand Volumes to Demand Flow Rates

Input parameters were specified in the Facts section above. Equation 14-1 is used to convert demand volumes to flow rates under equivalent ideal conditions:

\[
v_i = \frac{V_i}{PHF \times f_{HV}}
\]

From Chapter 12, Basic Freeway and Multilane Highway Segments, the passenger car equivalent \( E_T \) for trucks in level terrain is 2.0.

For the freeway demand volume,

\[
Q_{F} = 3 \times 4,000 = 3 \times 4,779 = 14,337 pc/h
\]

\[
v_F = \frac{4,000}{0.90 \times 0.93} = 4,779 pc/h
\]

For the ramp demand volume,

\[
Q_{R} = 490 pc/h
\]

\[
v_R = \frac{490}{0.90 \times 0.971} = 561 pc/h
\]

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

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 14-8, for an isolated on-ramp on a six-lane freeway, Equation 14-4 is used:

\[
v_{12} = v_F \times P_{FM}
\]

\[
P_{FM} = 0.5775 + 0.000028L_M = 0.5775 + 0.000028(820) = 0.600
\]

\[
v_{12} = (4,779)(0.600) = 2,867 pc/h
\]

From Exhibit 14-18, the adjustment factor applied to this result to find the estimated flow rate in Lanes 2 and 3 is 1.12. Therefore,

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

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 \( 1.5 \times 3,211/2 = 2,408 pc/h/ln \).
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: Estimate the Capacity of the Ramp–Freeway Junction and Compare with Demand Flow Rates**

For this case, there are three simple checkpoints:

1. The principal capacity checkpoint is the total demand flow rate downstream of the merge, $4,779 + 561 = 5,340$ pc/h. From Exhibit 14-10, 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.

2. The ramp roadway capacity should also be checked by using Exhibit 14-12. 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 561 pc/h.

3. 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 $3,211 + 561 = 3,772$ pc/h, which is lower than the maximum desirable flow rate of 4,600 pc/h, shown in Exhibit 14-10.

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

The density in the ramp influence area is found by using Equation 14-22, except $v_{23}$ replaces $v_{12}$ because of the left-hand ramp placement:

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

$$D_S = 5.475 + 0.00734(561) + 0.0078(3,211) - 0.00627(820) = 29.5 \text{ pc/mi/ln}$$

From Exhibit 14-3, this is LOS D.

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

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 14-13, the following results are obtained:

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

$$M_S = 0.321 + 0.0039e^{(3.777/1,000)} - 0.002(820 \times 30 \times 1.00/1,000) = 0.443$$

$$S_R = FFS \times SAF - (FFS \times SAF - 42)M_S$$

$$S_R = (65)(1.00) - (65 \times 1.00 - 42)(0.443) = 54.8 \text{ mi/h}$$
\[ S_0 = FFS \times SAF - 0.0036(v_{0A} - 500) \]
\[ S_0 = (65)(1.00) - 0.0036(1,568 - 500) = 61.2 \text{ mi/h} \]
\[ S = \frac{v_{23} + v_{0A}N_0}{\left(\frac{\alpha}{S_R}\right)} = \frac{3,777 + (1,568)(1)}{\left(\frac{\alpha}{54.8}\right) + \left(\frac{1,568 \times 1}{61.2}\right)} = 56.5 \text{ mi/h} \]

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 of Chapter 14 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 information is available concerning this example problem. The example assumes no impacts of inclement weather or incidents.

1. Single-lane, right-hand on-ramp with an FFS of 40 mi/h;
2. Six-lane freeway (three lanes in each direction) with an FFS of 70 mi/h;
3. Level terrain for freeway and ramp;
4. 6.5% trucks on both freeway and ramp segments;
5. Peak hour factor = 0.87;
6. Drivers are regular users of the facility; and
7. 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 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 14-3, 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 14-10 and Exhibit 14-12, 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 14-22 defines the density in an on-ramp influence area as follows:

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

In this case

- $v_R = 0.10 \times v_F$
- $L_A = 1,000$ ft

Equation 14-22 and Exhibit 14-8 give the following:

- $v_{12} = v_F \times P_{FM}$
- $P_{FM} = 0.5775 + 0.000028L_A = 0.5775 + 0.000028(1,000) = 0.6055$
- $v_{12} = 0.6055v_F$

Substitution of these values into Equation 14-22 gives

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

$$D_R = 5.475 + 0.00734v_F + 0.00472v_F - 6.27$$

$$D_R = 0.005454v_F - 0.795$$

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

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.
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

\[ v_{FR} = 7,200 = v_F + 0.10v_F = 1.10v_F \]

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

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.

However, 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 are in terms of passenger cars per hour under equivalent ideal conditions. To convert them 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 6.5% trucks on both the ramp and the mainline. For level terrain (Chapter 12, Basic Freeway and Multilane Highway Segments), \( E_T = 2.0 \). Then

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

\[ f_{HV} = \frac{1}{1 + 0.065(2 - 1)} = 0.939 \]

Service volumes are obtained by multiplying service flow rates by the specified PHF, 0.87. These computations are illustrated in Exhibit 28-4.

<table>
<thead>
<tr>
<th>LOS</th>
<th>Service Flow Rate, Ideal Conditions (pc/h)</th>
<th>Service Flow Rate, Prevaling Conditions (SF) (veh/h)</th>
<th>Service Volume (SV) (veh/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1,979</td>
<td>1,979 × 0.939 × 1 × 1.858 = 1,858</td>
<td>1,858 × 0.87 = 1,616</td>
</tr>
<tr>
<td>B</td>
<td>3,813</td>
<td>3,813 × 0.939 × 1 × 3,580 = 3,580</td>
<td>3,580 × 0.87 = 3,115</td>
</tr>
<tr>
<td>C</td>
<td>5,280</td>
<td>5,280 × 0.939 × 1 × 4,958 = 4,958</td>
<td>4,958 × 0.87 = 4,313</td>
</tr>
<tr>
<td>D</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>E</td>
<td>6,545</td>
<td>6,545 × 0.939 × 1 × 6,146 = 6,146</td>
<td>6,146 × 0.87 = 5,347</td>
</tr>
</tbody>
</table>

The service flow rates and service volumes shown in Exhibit 28-4 are stated in terms of the approaching hourly 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:

\[
v_F = \frac{V_F}{PHF \times f_{HV}} = \frac{4,000}{0.87 \times 0.939} = 4,896 \text{ pc/h}
\]

The density is estimated by using Equation 14-22, 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,

\[
v_{12} = 0.6055 \times 4,896 = 2,965 \text{ pc/h}
\]

Then, by using Equation 14-22,

\[
D_R = 5.475 + 0.00734v_R + 0.0078v_{12} - 0.00627L_A
\]

\[
D_R = 5.475 + 0.00734v_R + 0.0078(2,965) - 0.00627(1,000)
\]

\[
D_R = 22.33 + 0.00734v_R
\]

\[
v_R = \frac{D_R - 22.33}{0.00734}
\]

It is clear from this equation that neither LOS A \((D_R = 10 \text{ pc/mi/ln})\) nor LOS B \((D_R = 20 \text{ pc/mi/ln})\) can be achieved with a freeway demand flow of 4,896 pc/h.

For LOS C and D,

\[
v_R (\text{LOS C}) = \frac{28-22.33}{0.00734} = 772 \text{ pc/h}
\]

\[
v_R (\text{LOS D}) = \frac{35-22.33}{0.00734} = 1,726 \text{ pc/h}
\]

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

\[
v_{FD} = 7,200 - 4,896 + v_R
\]

\[
v_R (\text{LOS E}) = 7,900 - 4,896 = 3,004 \text{ pc/h}
\]

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 factor (0.939 from Case 1). Service flow rates are converted to service volumes by multiplying by the PHF. These computations for ramp service volumes are illustrated in Exhibit 28-5.
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.
Chapter 14, Freeway Merge and Diverge Segments, described a methodology for analyzing ramps and ramp junctions to estimate capacity, speed, and density as a function of traffic demand and geometric configuration. This chapter includes two supplemental problems that examine situations that are beyond the scope of the Chapter 14 methodology. A typical microsimulation-based tool is used for this purpose, and the simulation results are compared, where appropriate, with those of the HCM.

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 a high-occupancy vehicle (HOV) lane.

The need to determine performance measures based on the analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, the examples presented in this chapter have applied existing versions of alternative tools and therefore do not reflect the trajectory-based measures described in Chapter 7.

For purposes of illustration, the default calibration parameters of the simulation tool (e.g., lane-changing behavioral characteristics) were applied to these examples. However, most simulation tools offer the ability to adjust these parameters. The parameter values can have a significant effect on the results, especially when the operation is close to full saturation.

**PROBLEM 1: RAMP-METERING EFFECTS**

This problem analyzes the impacts of ramp metering along the segment. The HCM procedure for ramp-merge junctions cannot estimate the impacts of ramp metering. These impacts can be approximated to some extent by not allowing the ramp demand to exceed the ramp-metering rate. To address ramp metering at a more detailed level, a typical microsimulation tool was used to evaluate the impacts of ramp metering on the density and capacity of the merge.

The subject segment consists of an on-ramp followed by an off-ramp, separated by 1,300 ft. The upstream segment is 1 mi long. Each simulation run was for 1 full hour. It was assumed that the mainline demand was 6,111 veh/h and that the ramp demand was 444 veh/h. The ramp metering is clock-time based (i.e., the metering rate does not change as a function of the mainline demand).

Experiments were conducted to obtain the density and capacity of the subject segment as a function of the ramp-metering rate. The queue length upstream of the ramp meter was also obtained as a function of the ramp-metering rate. Exhibit 28-6 provides a graphics capture of the simulated site.
Exhibit 28-7 provides the density of the segment between the on-ramp and the off-ramp as a function of the ramp-metering rate (or discharge headway from the on-ramp). As shown, the density is not much affected by the ramp-metering rate. As expected, the density of Lane 1 (the rightmost lane) is the highest, while the density in Lane 4 is the lowest.

Exhibit 28-8 provides capacity as a function of the ramp-metering headway and when no ramp metering is implemented. As shown, the simulation model predicts that capacity is higher when ramp metering is implemented. Capacity in simulation is typically measured in the form of maximum throughput downstream of a queued segment and is therefore one of the outputs of the simulation, as opposed to an input as in the HCM.
Exhibit 28-9 provides the queue length expected on the ramp as a function of the ramp-metering headway and when no ramp metering is implemented. As expected, the queue length is higher when ramp metering is implemented, and it increases dramatically when the ramp-metering rate exceeds 8 s/veh. The reason for this increase is that the demand on the ramp is approximately 8 s/veh (444 veh/h corresponds to an average headway of 8.1 s/veh).
As indicated above, the effects of ramp metering cannot be evaluated with the HCM. The freeway facilities methodology (HCM Chapter 10) can handle changes in segment capacity; however, other tools are required to estimate what the maximum throughput would be under various types of ramp-metering algorithms and rates. Also, the HCM cannot estimate the queue length on the on-ramp as a function of ramp metering. An analytical method could be developed to estimate queue length as a function of demand and service rate at the meter.

PROBLEM 2: CONVERSION OF LEFTMOST LANE TO AN HOV LANE

This problem is also based on this chapter’s Example Problem 3. It evaluates operating conditions when the leftmost lane of the mainline is converted into an HOV lane. Exhibit 28-10 provides a graphics capture of the segment.

Exhibit 28-10
Graphics Capture of the Segment with an HOV Lane

Exhibit 28-11 and Exhibit 28-12 show the density and capacity of the ramp junction as a function of the percentage of carpools. As shown, when the percentage of carpools increases, the density of the HOV lane and the overall link capacity increase. This occurs because for the range of values tested here, the utilization of the HOV lane increases, which improves the overall link performance.

Exhibit 28-11
Density of a Ramp Junction as a Function of the Carpool Percentage
Exhibit 28-13 presents the density as a function of HOV violators, while Exhibit 28-14 presents the corresponding capacity. These two graphs assume that there are 10% carpools in the traffic stream. As shown, density generally decreases while capacity increases as the percentage of HOV violators increases. The reason is that under this scenario, the facility is more efficiently utilized as violations increase with general traffic using the HOV lane.
Exhibit 28-15 and Exhibit 28-16 present the density and capacity of the ramp junction as a function of the distance at which drivers begin to react to the presence of the HOV lane (i.e., the distance to the regulatory sign). As shown, the longer that distance, the lower the density of the HOV lane and the higher the density in the other lanes. The reason is that under this scenario the percentage of carpools is relatively low (10%). When the HOV lane begins, non-HOVs congregate in the remaining lanes. Capacity is reduced as the distance at which drivers begin to react increases, because the HOV lane is not utilized as much when drivers are given early warning to switch lanes.
Exhibit 28-17 and Exhibit 28-18 present the density and capacity of the ramp junction as a function of the percentage of HOV usage. As expected, when usage of the HOV lane increases, the density of the HOV lane and the overall link capacity increase.
The type of analysis presented in this example cannot be conducted with the HCM, since the method does not estimate the HOV lane density separately. Variables such as the impact of the distance of the HOV regulatory sign cannot be evaluated, since they pertain to driver behavior attributes and their impact on density and capacity. The impact of the percentage of carpools and the percentage of violators could perhaps be estimated with appropriate modifications of the existing HCM method.