CHAPTER 10

Capacity and Level of Service

Determination of the capacities of transportation systems and facilities is a major issue in the analysis of transportation flow. The capacity of a transportation system or facility is defined as the maximum number of vehicles, passengers, or the like, per unit time, which can be accommodated under given conditions with a reasonable expectation of occurrence.

Capacity is independent of demand in the sense that it does not depend on the total number of vehicles (or whatever) demanding service. It is expressed in terms of units of some specific thing, however, so that it does depend on traffic composition (for instance, for highways, the percentage of trucks or other heavy vehicles; or for airport runways, the percentage of heavy jet aircraft). It is dependent on physical and environmental conditions, such as the geometric design of facilities or the weather.

Finally, capacity is a probabilistic measure. There is some variation from time to time and place to place in the maximum number of units of transportation demand that can be accommodated by similar facilities. Not all of these variations can be accounted for by the normal determinants of capacity. The number quoted as the capacity of a facility represents a value with a reasonable expectation of occurrence, but may be exceeded on occasion. Moreover, it is to be expected that there will be random variations in the number of vehicles that can be accommodated over very short time intervals, so that capacity is often best thought of as the maximum average flow rate that can be sustained indefinitely, so long as there is no lack of demand.

A concept closely related to capacity and often confused with it is that of service volume or service flow rate. A service volume is the maximum number of vehicles, passengers, or the like, which can be accommodated by a given facility or system under given conditions at a given level of service. Although levels of service are defined somewhat differently, depending on the situation, they are always intended to relate the quality of traffic service to given volumes (or flow rates) of traffic. Levels of service may be based on such things as travel times (or speeds), total delay, probability of delay, comfort, safety, and so forth.
As discussed in Chapter 9, flow is the reciprocal of the time separation, or headway, between vehicles. The maximum flow rate or capacity, then, is the reciprocal of the minimum average headway that can be attained under given conditions. That is,

\[ C = \frac{1}{\bar{h}_{\text{min}}} \]  

(10.1)

where \( C \) represents capacity and \( \bar{h}_{\text{min}} \) represents the minimum average headway. The minimum average headway, in turn, depends on the headway distribution, the speed distribution, and the degree of maneuverability in the traffic stream. Minimum headways may be determined deliberately and imposed upon a whole system by a controller, as in the case of air or rail systems, or they may result from the behavior of individual operators, as in the case of highway traffic. Actual minimum headways vary greatly, depending on the type of control system, the ability of vehicles to make emergency maneuvers, and the consequences of accidents. For highway traffic, average minimum headways may sometimes be as small as 1.5 s/vehicle. For rail transit systems, minimum headways are around 10 to 20 s/train. Air traffic headways are expressed in distance rather than time, with minimum spacings on final approach paths varying from 3 to 5 nautical miles (NM).

The degree to which theoretical minimum headways can be attained throughout a traffic stream depends on the speed distribution and maneuverability of vehicles. Unless the speed distribution is absolutely uniform (that is, all vehicles travel at the same speed) vehicles must be able to pass one another, or else gaps will develop in the traffic stream. Such gaps will mean that the capacity implied by the minimum headway value cannot actually be attained.

Different transportation systems vary a great deal with respect to the uniformity of speed distributions and the degree of maneuverability. At one extreme, rail rapid transit systems ideally operate at uniform speeds with no maneuverability; at the other, air traffic (except on runway approaches) exhibits a wide range of speeds and almost complete maneuverability in three dimensions. Highway traffic falls somewhere in between. In the case of highway traffic, moreover, there is a definite relationship between maneuverability and the uniformity of the speed distribution: as traffic volumes increase, maneuverability decreases and speeds become more uniform. Nevertheless, considerable maneuverability still exists up to the point of flow breakdown at capacity.

In situations in which maneuverability is restricted and nonuniform speed distributions are present, factors other than the minimum time headway determine capacity. This is the case, for instance, in mixed rail operations, in which the frequency and length of sidings (or the frequency of crossovers for double tracked lines) largely determine the capacity of the line, and on runway approach paths, where there is essentially no maneuverability and capacity is greatly dependent on the speed distribution.

### 10.1 AIR TRAFFIC CAPACITY

An airport can be thought of as a system composed of numerous parts, any one of which can serve as a bottleneck.\(^1\) Airports are commonly divided for purposes of analysis into an air side, which includes terminal airspace, common approach paths for
10.1 Air Traffic Capacity

runways, runways, taxiways, and aprons, and a ground side, which includes facilities such as terminal buildings, gates, parking facilities, and baggage handling systems. The present discussion will be concerned with the air side only.

Normally, the air side bottleneck at an airport will be either the runways or the common approach paths to the runways. Critical factors in determining the capacity of the runway system are the amount of time aircraft spend on the runway and the time separations of the aircraft on the common approach path. Air traffic rules state that there can be only one aircraft on the runway at a time and that minimum distance separations of 3 NM must be maintained behind conventional aircraft, 4 NM where a heavy jet aircraft is following another heavy jet, and 5 NM where a conventional aircraft is following a heavy jet.

The capacity of a runway is the reciprocal of the average service time for aircraft using it. The service time, in turn, is determined by either the runway occupancy time or the time separation of aircraft at the runway threshold. In practice, time separations at the runway threshold are more likely to be critical than are runway occupancy times.

In Section 8.1, a space–time diagram was used to derive expressions for interarrival times at the runway threshold, based on the speeds of the leading and trailing aircraft, the minimum distance separation, and the length of the common approach path. To repeat these,

\[
t_{ij} = \begin{cases} 
\frac{\delta}{v_j} & \text{for } v_i \leq v_j \\
\frac{\delta}{v_j} + \gamma \left( \frac{1}{v_j} - \frac{1}{v_i} \right) & \text{for } v_i > v_j
\end{cases}
\]  

(10.2)

where

\[v_i = \text{speed of lead aircraft}\]
\[v_j = \text{speed of trailing aircraft}\]
\[\delta = \text{minimum distance separation}\]
\[\gamma = \text{length of common approach path}\]

In the case of a runway used for arrivals only, runway capacities may be determined by calculating the weighted average interarrival time for all aircraft using the runway. This is done by first using Equation (10.2) to determine ideal interarrival times for each pair of aircraft classes using the runway. Actual average interarrival times will normally be greater, because Equation (10.2) leaves no room for error on the part of the air traffic controller who is responsible for ensuring that the minimum distance separations will be maintained. To account for the controller’s uncertainty, a buffer time is added to each time separation as calculated by Equation (10.2). This buffer is determined by holding the probability of a violation of the air traffic rules to some stated level, based on certain assumptions about the distribution of actual aircraft positions around the estimated position. Once these corrected interarrival times have been calculated, their weighted average is found by the formula

\[
\overline{\tau}_{\text{min}} = \sum_i \sum_j p_{ij} t_{ij}
\]  

(10.3)

where \(p_{ij}\) is the probability of aircraft pair \(ij\). If the order of arrival of aircraft is random,

\[
p_{ij} = p_i p_j
\]  

(10.4)
Once the weighted average time separation $T_{\text{min}}$ is determined, the capacity is its reciprocal.

**EXAMPLE PROBLEM 10.1** Determine the capacity of an airport runway handling arrivals only that is used by the following aircraft classes:

<table>
<thead>
<tr>
<th>Class</th>
<th>Type</th>
<th>Speed, knots</th>
<th>Percent of traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Conventional</td>
<td>120</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>Conventional</td>
<td>150</td>
<td>60</td>
</tr>
</tbody>
</table>

Buffer times are 33 s for each aircraft pair except pair 2–1, for which the buffer time is 15 s. The common approach path is 6 NM long. Since both aircraft are conventional, all distance separations are 3 NM.

Find ideal interarrival times:

$$t_{11} = \frac{3}{120} \times (3,600 \text{ s/h}) = 90 \text{ s}$$
$$t_{12} = \frac{3}{150} \times (3,600 \text{ s/h}) = 72 \text{ s}$$
$$t_{21} = \frac{3}{120} + 6\left(\frac{1}{120} - \frac{1}{150}\right)(3,600 \text{ s/h}) = 126 \text{ s}$$
$$t_{22} = \frac{3}{150} \times (3,600 \text{ s/h}) = 72 \text{ s}$$

Ideal interarrival times are

<table>
<thead>
<tr>
<th>Trailing aircraft</th>
<th>Lead aircraft</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>90</td>
<td>126</td>
</tr>
<tr>
<td>2</td>
<td>72</td>
<td>72</td>
<td>72</td>
</tr>
</tbody>
</table>

Buffers are

<table>
<thead>
<tr>
<th>Trailing aircraft</th>
<th>Lead aircraft</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>33</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>33</td>
<td>33</td>
<td>33</td>
</tr>
</tbody>
</table>

Adding buffer times to ideal interarrival times, total interarrival times are

<table>
<thead>
<tr>
<th>Trailing aircraft</th>
<th>Lead aircraft</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>123</td>
<td>141</td>
</tr>
<tr>
<td>2</td>
<td>105</td>
<td>105</td>
<td>105</td>
</tr>
</tbody>
</table>
10.2 Rail Capacity

Weighted average interarrival time:
\[ \bar{t}_{ij} = (0.4)(0.4)(123) + (0.4)(0.6)(141) + (0.6)(0.4)(105) \\
+ (0.6)(0.6)(105) = 116.52 \]

Capacity:
\[ C = \frac{1}{\bar{t}_{\text{min}}} = \frac{1}{116.52} \times 3600 \text{ s/h} = 30.89 \approx 31 \text{ aircraft/h} \]

More commonly, runways handle *mixed operations*, that is, both arrivals and departures. Under these conditions, arrivals have priority over departures; only one aircraft is allowed to be on the runway at a time; and a departure may not be released if an arrival is within a specified distance of the runway threshold, usually 2 NM. Space–time diagrams may be constructed showing the sequence of operations implied by these rules and from these, analytical expressions may be derived to give the minimum interarrival times required to release a given number of departures between successive arrivals. Assuming equal numbers of arrivals and departures, the interarrival times required to release one departure between each pair of arrivals would be calculated. For similar aircraft mixes, these will be greater than the interarrival times for runways used for arrivals only.

Where there is only one runway, mixed operations are necessary. Even where there are multiple runways, mixed operations will usually provide higher overall capacities than will operations in which arrivals and departures are segregated, since the interarrival times for mixed arrivals are usually much less than twice those for arrivals only. Finally, where there are multiple runways, operations on the various runways will interfere with one another to some extent. Since the degree of interference depends on the exact runway configuration, airport capacity also depends on the runway configuration.

**10.2 RAIL CAPACITY**

Methods for determining the capacity of rail lines depend on the type of rail line (whether single or double-tracked), the speed distribution of trains, and the type of control system employed. The simplest rail capacity problem is that involving rail rapid transit systems. These usually have the following characteristics:

- One-way operation.
- A common speed profile for all trains. That is, each train traverses each section of track at the same speed as all other trains; consequently, trains do not overtake or pass one another.
- Common station dwell times. That is, each train spends the same amount of time stopped at each station as all other trains; dwell times may vary from station to station, however.
- A fixed minimum front-to-back time gap between trains.
Given these characteristics, the capacity problem for rail rapid transit is one of determining the effective front-to-front headways between trains. Since all trains stop in the stations, and trains cannot enter the station until the preceding train has cleared it, the critical time separation is that required at the stations.

Figure 10.1 is a space-time diagram showing the arrival of successive trains at a station. In the diagram, the vertical axis is time and the horizontal axis is distance. The vertical lines represent the boundaries of the station. The time $h$, represents the total front-to-front headway; $t_g$, the minimum front-to-back time gap between trains; $t_s$, the station dwell time; and $t_a$ and $t_d$, times consumed in starting and stopping the train, which depend on the lengths of the trains and the acceleration and deceleration rates. The diagram shows the trajectories of the front and rear of the first train as it enters and leaves the station; a dashed line offset by a vertical distance $t_g$ from the trajectory of the rear of the first train, which indicates the space-time region that the second train must avoid in order to not violate the minimum time gap; and the trajectory of the second train as it enters the station. Clearly,

$$h = t_g + t_s + t_a + t_d$$

and the capacity of the line is

$$C = \frac{1}{h}$$
Note that, where dwell times vary from station to station, the capacity of the line depends on the maximum dwell time.

It should also be noted that rail transit systems are more concerned with the capacities of their lines in terms of passengers than they are in their capacities in terms of trains. This means that the length of the trains is also important. From Figure 10.1, it is evident that train length does influence times $t_a$ and $t_d$, but these are only a minor part of the headway. On the other hand, the passenger-carrying capacity of a train increases directly with its length. This means that the passenger capacity of a rail transit line is reached when maximum-size trains are used. Train size, in turn, is usually limited by the length of station platforms.

The case of a freight (or freight-passenger) rail line with one-way mixed speed operations is somewhat more complex. In this case, trains do overtake and pass one another. Since passing can occur only where there are crossovers or sidings, each rail line will have a unique capacity depending on the spacing of crossovers or sidings. In order to analyze a specific line, it is necessary to also be able to compute minimum headways on sections between sidings or crossovers. These headways depend on the control system of the railway, which may be simulated as outlined in Section 8.1. As a general rule, the more complex the block signal control system (that is, the more aspects or speed levels involved) and the shorter the blocks, the shorter the minimum headways and, consequently, the greater the line capacity.

Single-track railways with two-way operations also employ block-signal control systems. In this case, the block-signal control system activates signals on both sides of the occupied block to provide protection against oncoming trains. For single-track lines, the really critical factors are the spacing and length of the sidings and the efficiency of the dispatching policy, since trains traveling in opposite directions must pass one another in sidings. Once again, each line poses a unique capacity problem.

Because maneuverability on rail lines is so restricted, rail capacity tends to be partly a dispatching problem under any circumstances. Modern railways use central traffic control (CTC) systems to improve capacities by scheduling trains in such a way as to minimize conflicts and delays. Central traffic control systems will normally involve a single dispatching center for an entire system. Train detection and communications systems allow dispatchers in this center to control all trains on the system. Railroad operating experience suggests that central traffic control systems can make a substantial difference in rail line capacities. Typically, implementation of a central control system on a single-track line with a block signal control system will increase line capacity from 30 to 65 trains/day; on a double-track line, the corresponding increase will be from 60 to 120 trains/day.

Like rail transit systems, however, freight railroads are rarely concerned with line capacity in terms of trains per unit time. Rather, they are concerned with the tonnages that can be hauled over a relatively long period of time. On heavily utilized sections of track, the real restriction is that track maintenance interferes with track utilization; the need to maintain the track establishes an upper limit to the number of hours per day that the track can be used, and this, rather than the capacity in the usual sense, restricts the tonnages that can be hauled. Also, the capacity of rail line in terms of tonnage depends more on train lengths and car sizes than on the number of trains per unit time.
10.3 HIGHWAY CAPACITY

Capacities of airports and rail systems are largely functions of their control systems. Highway systems, by contrast, involve very little positive control; as a result, their capacities and other flow characteristics depend heavily on driver behavior. The analysis of highway capacity is based primarily on empirical relationships, such as the speed-flow and flow-density relationships introduced in Section 9.2. Methods for analyzing capacities and service flow rates based on these empirical relationships are incorporated in the *Highway Capacity Manual (HCM)* published by the Transportation Research Board (TRB). The latest edition of the *HCM* is published in two versions, one in metric units and the other in customary units. The material in this section is based on the metric version; Appendix C contains exhibits (tables and charts) for the metric version and Appendix D contains the equivalent exhibits in customary units. In general, procedures are the same for both versions. Numbers in Appendix D are hard-converted from the metric version and will give slightly different results from those based on the metric version. TRB advises that analyses should be conducted entirely in one system of units and conversions between systems of units should not be made.

Highway systems are composed of a number of different elements, any one of which can limit their capacity. In the case of freeways, these include the basic freeway segment itself, onramp and offramp junctions, and weaving sections. In the case of urban arterial streets, the capacity of signalized intersections is usually the controlling factor. For other classes of roadway, such as rural two-lane highways or multilane non-freeways, either the basic roadway segment or intersections may be critical. Although the *Highway Capacity Manual* deals with all of these situations, and several others, the present discussion will be confined to four chapters from the *Manual*: Chapter 23, *Basic Freeway Segments*; Chapter 25, *Ramps and Ramp Juncions*; Chapter 20, *Two-Lane Rural Highways*; and Chapter 16, *Signalized Intersections*.

10.3.1 Basic Freeway Segments

The *Highway Capacity Manual* recognizes three critical elements of freeway systems—basic freeway sections, ramps and ramp junctions, and weaving sections—and presents different techniques for analyzing their capacity. Figure 10.2 illustrates these different elements.

Basic freeway segment capacity is the simplest highway capacity problem covered by the *Manual*. Conditions on basic freeway segments are essentially uninterrupted one-way flow. Under these conditions, capacity (defined by the *HCM* as the maximum flow that can be sustained for 15 min) is stated by the *Manual* to vary with the free-flow speed of the freeway, and to range from 2,250 passenger cars per hour per lane (pc/h/ln) for a free-flow speed of 90 km/h up to 2,400 pc/h/ln for free-flow speeds of 120 km/h or more. These numbers are stated to be capacities under ideal conditions, which include 3.6 m lanes, minimum right shoulder clearances of at least 1.8 m, all passenger cars, 10 or more lanes (applies to urban areas only), interchanges spaced...
every 3.3 km or more, level terrain (no grades greater than 2 percent), and a driver population consisting of regular and familiar users of the facility. In cases in which these conditions are not met, capacities will be decreased.

The *Highway Capacity Manual* defines six levels of service, designated by the letters A through F, with A being the highest level of service and F the lowest. The definitions of these levels of service vary depending on the type of roadway or roadway element under consideration. In the case of basic freeway sections, the levels of service are based on density, and are given in Table 10.1 (also Table C.1). Level of service F

<table>
<thead>
<tr>
<th>Level of service</th>
<th>Density, pc/km/ln</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0–7</td>
</tr>
<tr>
<td>B</td>
<td>7–11</td>
</tr>
<tr>
<td>C</td>
<td>11–16</td>
</tr>
<tr>
<td>D</td>
<td>16–22</td>
</tr>
<tr>
<td>E</td>
<td>22–28</td>
</tr>
<tr>
<td>F</td>
<td>&gt;28</td>
</tr>
</tbody>
</table>

represents congested flow. Speed and flow are also related to densities and may be related, in turn, to the various levels of service. Table C.2 gives approximate limiting values of speeds, flow rates, and volume/capacity ratios for the various levels of service. Figure 10.3 (also Figure C.1) is a set of speed-flow curves showing the relationship between speed, flow, and level of service.

Determination of the level of service for a basic freeway section involves determination of the free-flow speed, the 15-min flow rate, and the level of service. Figure 10.4 is a flow chart illustrating the procedure.

The free-flow speed may be determined by either a field study, in which speeds are measured for low to moderate traffic volumes (up to 1,300 pc/h/ln) or by the following formula

\[
FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}
\]  

(10.7)

where

- \(FFS\) = estimated free-flow speed, km/h
- \(BFFS\) = base free-flow speed, 110 (urban) or 120 km/h (rural)
- \(f_{LW}\) = adjustment for lane width from Table C.3, km/h
- \(f_{LC}\) = adjustment for right shoulder clearance from Table C.4, km/h
- \(f_N\) = adjustment for number of lanes from Table C.5, km/h
- \(f_{ID}\) = adjustment for interchange density from Table C.6, km/h

The 15-min flow rate in pc/h/ln is calculated from the hourly volume of mixed traffic by

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

(10.8)
where \( v_p \) = 15-min passenger car equivalent flow rate, pc/h/ln
\( V \) = hourly volume, veh/h
PHF = peak hour factor
\( N \) = number of lanes in one direction
\( f_{HV} \) = heavy-vehicle factor
\( f_p \) = driver population factor

The peak hour factor (PHF) is defined as the ratio of the hourly volume to the peak 15-min flow rate.
The heavy vehicle adjustment factor $f_{HV}$ is calculated as follows. The HCM identifies two classes of heavy vehicles: trucks and buses (considered to be equal in their impact on traffic flow) and recreational vehicles. Each heavy vehicle is thought of as being equivalent to some number of passenger cars. These passenger car equivalents vary with the type of heavy vehicle, the percentage of heavy vehicles in the traffic stream, and the length and severity of grades. They are defined for both extended general freeway segments, for which terrain is classified as level, rolling, or mountainous (Table C.7) and for specific combinations of length and severity of upgrade (Tables C.8 and C.9) and downgrade (Table C.10). The factor $f_{HV}$ may be computed from individual passenger car equivalents values as follows:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \quad (10.9)$$

where $E_T, E_R =$ passenger car equivalents for trucks and buses and for recreational vehicles, respectively

$P_T, P_R =$ proportion of trucks and buses and of recreational vehicles, respectively, in the traffic stream

The adjustment factor for driver population $f_p$ ranges from 1.00 to 0.85. For urban weekday or commuter traffic, $f_p$ is taken to be 1.0; lower values may be used where evidence exists that capacity is reduced as a result of the presence of drivers unfamiliar with the roadway, for instance, in recreational areas.

The level of service for basic freeway sections may be determined from Figure 10.3 and Table C.1 or C.2. Using the measured or estimated free-flow speed, a speed-flow curve may be constructed with the same shape as those in Figure 10.3. From this curve and $v_p$, the estimated speed $S$ is determined. Alternatively, $S$ may be calculated as follows:

For $90 \leq FFS \leq 120$ and $(3,100 - 15FFS) < v_p \leq (1,800 + 5FFS)$,

$$S = FFS - \left[ \frac{1}{28} (23FFS - 1,800) \left( \frac{v_p + 15FFS - 3,100}{20FFS - 1,300} \right) \right]^{2.6} \quad (10.10)$$

For $90 \leq FFS \leq 120$ and $v_p \leq (3,100 - 15FFS)$,

$$S = FFS \quad (10.11)$$

The density $D$ is then calculated as

$$D = \frac{v_p}{S} \quad (10.12)$$

and compared with the limiting values in Table C.1 or C.2. As an alternative, maximum service flow rates for speeds falling between those in Table C.2 may be determined by interpolation, and the level of service determined by comparing $v_p$ with these limiting values.
EXAMPLE PROBLEM 10.2 A rural freeway has an ideal free-flow speed of 120 km/h and two 3.6 m lanes in each direction, with right shoulder lateral clearance of 1.2 m. Interchanges are spaced approximately 5 km apart. Traffic consists of 10 percent trucks and buses and 8 percent recreational vehicles. The adjustment for driver population factor is estimated to be 0.80. If the maximum 15-min flow rate is 1,760 veh/h, what is the level of service on a 1.7 km long 3.1 percent upgrade?

Determine free-flow speed:

\[
FFS = BFFS - f_{LW} - f_{LC} - f_{N} - f_{ID}
\]

\[
f_{LW} = 0.0
\]

\[
f_{LC} = 1.9
\]

\[
f_{N} = 0.0 \text{ (See note at bottom of Table C.5)}
\]

Interchanges per kilometer = \( \frac{1}{5} = 0.20 \)

\[
f_{ID} = 0.0
\]

\[
FFS = 120.0 - 0.0 - 1.9 - 0.0 - 0.0 = 118.1 \text{ km/h}
\]

Determine adjustment factors:

Driver population factor:

\[
f_{p} = 0.80 \text{ (given)}
\]

Heavy vehicle factor:

\[
E_{T} = 3.0 \text{ (Table C.7)}
\]

\[
E_{R} = 2.0 \text{ (Table C.8)}
\]

\[
f_{HV} = \frac{1}{1 + 0.10(3.0 - 1) + 0.08(2.0 - 1)} = \frac{1}{1.28} = 0.78
\]

Number of lanes:

\[
N = 2 \text{ (given)}
\]

Determine \( v_{p} \):

\[
v_{p} = \frac{V}{PHF \times N \times f_{HV} \times f_{p}}
\]

Flow is given as \( V/PHF \):

\[
v_{p} = \frac{1,760}{2 \times 0.78 \times 0.80} = 1,410 \text{ pc/h}
\]

\[
3,100 - 15FFS = 1,329
\]

\[
1,329 < 1,440 \text{ so}
\]
EXAMPLE PROBLEM 10.3 Generalized terrain for the freeway described in Example Problem 10.2 is rolling. How many lanes are required to provide level of service B? Free-flow speed is the same as in Example Problem 10.2.

Adjustment factors are the same as in Example Problem 10.2, except for heavy vehicle factor.

Heavy vehicle factor:

\[ E_T = 2.5 \]
\[ E_R = 2 \]
\[ f_{HV} = \frac{1}{1 + 0.10(2.5 - 1) + 0.08(2 - 1)} = \frac{1}{1.23} = 0.81 \]
\[ f_p = 0.80 \]

Interpolate, using values from Table C.2, to find maximum service flow rate at level of service B for 118.1 km/h:

\[ SF_B = 1.210 + \frac{118.1 - 110.0}{120.0 - 110.0} (1.320 - 1.210) = 1.299 \text{ pc/h} \]

Set \( v_p = 1.299 \text{ pc/h} \) and solve equation for \( N \):

\[ v_p = \frac{V}{N \times f_{HV} \times f_p} \]
\[ N = \frac{V}{v_p \times f_{HV} \times f_p} = \frac{1.760}{1.299 \times 0.81 \times 0.80} = 2.09 \]

Round up to 3 lanes

10.3.2 Ramps and Ramp Junctions

Capacity and level of service for freeway ramps are discussed in Chapter 25 of the *Highway Capacity Manual*. Normally, the critical elements for ramp capacity are the freeway and ramp roadways upstream and downstream of the ramp junction. The level of service, on the other hand, depends on the density in the outer two lanes.
of the freeway (lanes 1 and 2) in an area of influence that extends 450 m downstream of an onramp or 450 m upstream of an offramp.

Figure 10.5 illustrates critical features of onramp and offramp junctions. Note that the Manual assumes that acceleration and deceleration lanes are present. The Manual defines the acceleration lane length $L_A$ and the deceleration lane length $L_D$ as being from the point at which the left lane of the ramp and the right lane of the freeway converge to the end of the taper connecting the acceleration or deceleration lane to the freeway. Provision of acceleration or deceleration lanes is not a universal practice; however, because of the way the lane lengths are defined, some distance $L_A$ or $L_D$ will always exist, even for continuously tapered ramp junctions. Where ramp junctions are continuously tapered, $L_A$ will usually be about 180 m and $L_D$ will be about 45 m.

Figure 10.6 is a flowchart illustrating the procedure for calculating level of service for ramps and ramp segments. In order to check capacities and determine levels of service for ramp junctions, it is first necessary to convert all flows to peak 15-min flow rates in passenger car equivalents and to adjust them for lane width and driver populations. The overall conversion is as follows, where the various factors are the same as for basic freeway segments and where $v_i$ is the peak 15-min flow in passenger cars per hour and $V_i$ is the hourly volume in mixed vehicles per hour:

$$v_i = \frac{V_i}{PHF_{HV}f_p}$$  \hspace{1cm} (10.13)\n
Once peak flow rates in passenger cars per hour are determined, the fraction of the total freeway flow in lanes 1 and 2 is determined. Tables C.11 and C.12 give equations for calculating the fraction of flow in lanes 1 and 2 for onramps and offramps, respectively. Variables used in these equations are as follows:

$P_{FM}$ = fraction of freeway flow in lanes 1 and 2 immediately upstream of merge

$P_{FD}$ = fraction of freeway flow in lanes 1 and 2 immediately upstream of diverge

$L_A$ = length of acceleration lane, m

$L_D$ = length of deceleration lane, m
266 CHAPTER 10: Capacity and Level of Service

Input
- Geometric data
- Ramp free-flow speed
- Demand

Demand Flow Adjustment
- Peak-hour factor
- Heavy vehicle factor
- Driver population factor

Compute demand flow rate immediately upstream of the diverge influence area
- Lanes 1 and 2 of the mainline

Compute demand flow rate immediately upstream of the merge influence area
- Lanes 1 and 2 of the mainline

Compute Capacity
- Total flow departing diverge area
- Maximum flow entering Lanes 1 and 2 prior to deceleration lane
- Existing legs of the freeway

Compute Capacity
- Total flow leaving merge area
- Maximum flow entering merge area

Compute density
Determine level of service
Compute speeds

FIGURE 10.6
Ramps and ramp junctions methodology.

\[ u_F = \text{total freeway demand flow upstream of ramp} \]
\[ u_R = \text{ramp demand flow rate} \]
\[ u_U = \text{flow on upstream ramp} \]
\[ u_D = \text{flow on downstream ramp} \]
\[ L_{down} = \text{distance to downstream ramp (ramp nose to ramp nose)} \]
\[ L_{up} = \text{distance to upstream ramp (ramp nose to ramp nose)} \]
\[ S_{FR} = \text{free-flow speed on ramp as it approaches freeway} \]
Tables C.13 and C.14 are keys for selecting the appropriate equation for given conditions. Note that for four-lane freeways, the calculation of \( P_{FM} \) and \( P_{FD} \) is trivial, since there are only two lanes in one direction. Note also that in the cases of onramps to six-lane freeways with upstream or downstream offramps and offramps to six-lane freeways with upstream onramps or downstream offramps, two or three different equations may apply. For onramps, if the choice is between Equations 1 and 2, Equation 1 should be used if \( L_{up} \) is greater than or equal to the equilibrium distance \( L_{EQ} \), where

\[
L_{EQ} = 0.0675(v_F + v_R) + 0.46L_A + 10.24S_{FR} - 747 \quad (10.14)
\]

If the choice is between Equations 1 and 3, Equation 1 is used if \( L_{EQ} \) is greater than \( L_{down} \), where

\[
L_{EQ} = \frac{V_U}{0.2337 + 0.000076V_F - 0.00025v_R} \quad (10.15)
\]

Where both adjacent upstream and downstream offramps exist, values of \( P_{FM} \) should be calculated for all cases, and the largest value is used. For offramps, where the choice is between Equations 5 and 6, \( L_{EQ} \) is given by

\[
L_{EQ} = \frac{v_D}{3.79 - 0.000658(v_F + v_R)} \quad (10.16)
\]

and Equation 5 is used if \( L_{down} \geq L_{EQ} \). Where the choice is between Equations 5 and 7, \( L_{EQ} \) is given by

\[
L_{EQ} = \frac{v_D}{0.3596 + 0.001149L_A} \quad (10.17)
\]

and Equation 5 is used if \( L_{down} \geq L_{EQ} \). If both an adjacent downstream offramp and an adjacent upstream onramp exist, values of \( P_{FD} \) are calculated by all applicable equations and the greatest is used.

For onramps, flow in lanes 1 and 2 is given by

\[
v_{12} = v_F \times P_{FM} \quad (10.18)
\]

For offramps, flow in lanes 1 and 2 is given by

\[
v_{12} = v_R + (v_F - v_R)P_{FD} \quad (10.19)
\]

Once the flow in lanes 1 and 2 has been calculated, the applicable flow rates may be compared with the capacities and maximum desirable flows entering the influence area that are given by Tables C.15 through C.17.

In the case of merges, the critical freeway flow will normally be that downstream of the ramp junction, given by

\[
v_{FO} = v_F + v_R \quad (10.20)
\]

This is compared with the limiting values in Table C.15. If the limiting value for the appropriate number of lanes and free-flow speed is exceeded, the merge exceeds capacity and the level of service is F. If not, the flow in lanes 1 and 2 immediately
downstream of the merge, given by

$$v_{R12} = v_R + v_{12}$$  \hspace{2cm} (10.21)

is compared with the last column of Table C.15 to determine whether flow in lanes 1 and 2 exceeds its maximum desirable value. If so, locally high densities are apt to occur, but no queuing is expected on the freeway. If $v_{R0}$ is less than the limiting value, the level of service will depend on the density in lanes 1 and 2 in the influence area, regardless of whether $v_{R12}$ exceeds its maximum desirable value.

In the case of diverges, the critical freeway flow is normally upstream of the ramp; however, it is also necessary to perform capacity checks for freeway flow downstream of the diverge (in cases in which the number of lanes decreases) and for $v_R$. This last check is important because diverge areas often fail because the offramp capacity is exceeded. Freeway flows are checked against capacities given in Table C.16 and ramp demand flow is checked against those in Table C.17. Note that this last check may not be adequate, since the capacity of the ramp roadway itself is less likely to limit capacity than is the capacity of an intersection at the ramp terminal. If any of these flows exceeds its limiting value, the level of service is F. If not, the flow entering the diverge area $v_{12}$ is compared with the last column of Table C.16 to determine whether it exceeds its maximum desirable value; as in the case of merge areas, failure of this check indicates the likelihood of high-density flow in this area, but does not indicate that the ramp junction is over capacity. As in the case of merges, if none of the capacity checks fails, the level of service will depend on the density in lanes 1 and 2 in the influence area.

For merge areas, the density in the ramp influence area is given by

$$D_R = 3.402 + 0.00456v_R + 0.0048v_{12} - 0.01278L_A$$  \hspace{2cm} (10.22)

For diverge areas, it is given by

$$D_R = 2.642 + 0.0053v_{12} - 0.0183L_D$$  \hspace{2cm} (10.23)

To determine the level of service, $D_R$ is compared with the limiting values given by Table C.18.

The procedures outlined above apply to single-lane ramps. The HCM also discusses modifications of these procedures for cases involving two-lane ramp junctions, left-hand entrances or exits, and freeways with more than four lanes in one direction.

**EXAMPLE PROBLEM 10.4** What is the level of service for the ramp combination and traffic conditions shown below? PHF = 0.87. The terrain is level. Free-flow speed on the freeway is 100 km/h and free-flow speed on the ramps is 70 km/h.
1. Convert flows to peak 15-min flow, pc/h:

   Freeway flow:
   \[ f_{HV} = \frac{1}{1 + p_T(E_T - 1)} = \frac{1}{1 + 0.04(1.5 - 1)} = 0.980 \]
   \[ v_i = \frac{V_i}{PHF_{fHV} f_p} = \frac{2.800}{(0.87)(0.980)(1.00)} = 3.284 \]

   Onramp flow:
   \[ v_i = \frac{V_i}{PHF_{fHV} f_p} = \frac{450}{(0.87)(0.980)(1.00)} = 528 \]

   Offramp flow:
   \[ f_{HV} = \frac{1}{1 + p_T(E_T - 1)} = \frac{1}{1 + 0.06(1.5 - 1)} = 0.971 \]
   \[ v_i = \frac{V_i}{PHF_{fHV} f_p} = \frac{600}{(0.87)(0.971)(1.00)} = 710 \]

2. Merge:

   Determine \( P_{FM} \) (Equation 3 or Equation 1):
   \[ L_{EQ} = \frac{v_D}{0.3596 + 0.001149L_A} = \frac{710}{0.3596 + 0.001149(250)} = 1,098 \text{ m} \]
   \[ 500 < 1,098 \quad \text{Use Equation 3} \]
   \[ P_{FM} = 0.5487 + \frac{0.0801v_D}{L_{down}} \]
   \[ = 0.5487 + \frac{(0.0801)(710)}{500} = 0.662 \]

   Determine \( v_{12} \):
   \[ v_{12} = v_F \times P_{FM} = (3.284)(0.662) = 2.174 \]

   Determine \( v_{FO} \) and \( v_{R12} \):
   \[ v_{FO} = v_F + v_R = 3.284 + 528 = 3.812 \]
   \[ v_{R12} = v_R + v_{12} = 528 + 2.174 = 2.702 \]

   Check capacity (Table C.15):
   \[ 3,812 < 6,900 \quad \text{under capacity} \]

   Check maximum flow in influence area:
   \[ 2,702 < 4,600 \quad \text{OK} \]

   Estimate density:
   \[ D_R = 3.402 + 0.00456v_R + 0.0048 v_{12} - 0.01278L_A \]
   \[ = 3.402 + 0.00456(528) + 0.0048(2.174) - 0.01278(250) = 13.05 \]
Determine level of service (Table C.18):

\[ 12 < 13.05 < 17 \quad \text{level of service C} \]

3. Diverge:

Determine \( P_{FD} \) (Equation 6 or Equation 5):

\[
L_{EQ} = \frac{v_U}{0.2337 + 0.000076v_F - 0.00025v_R} = \frac{528}{0.2337 + 0.000076(3,812) - 0.00025(710)} = 1,526 \text{ m}
\]

\[ 500 < 1,526 \quad \text{use Equation 6} \]

\[
P_{FD} = 0.717 - 0.000039v_F + \frac{0.184v_U}{L_{EQ}} = 0.717 - 0.000039(3,812) + \frac{(0.184)(528)}{500} = 0.763
\]

Determine \( V_{12} \):

\[
v_{12} = v_R + (v_F - v_R)P_{FD} = 710 + (3,812 - 710)(0.763) = 3,077
\]

Check capacity (Tables C.16 and C.17):

\[ 3,812 < 6,900 \quad \text{under capacity} \]

\[ 710 < 2,100 \quad \text{under capacity} \]

Check maximum flow in influence area:

\[ 3,077 < 4,400 \quad \text{OK} \]

Estimate density:

\[
D_R = 2.642 + 0.0053v_{12} - 0.0183L_D = 2.642 + 0.0053(3,077) - 0.0183(50) = 18.04
\]

Determine level of service (Table C.18):

\[ 17 < 18.04 < 22 \quad \text{level of service D} \]

\[ Diverge \text{ governs. Overall level of service is D} \]

**EXAMPLE PROBLEM 10.5** What is the level of service for the ramp combination and traffic conditions shown below? PHF = 0.90. The terrain is level. Free-flow speed on the freeway is 100 km/h and free-flow speed on the ramps is 70 km/h.

![Diagram of ramp combination and traffic conditions](image-url)
1. Convert flows to peak 15-min flow, pc/h:

Freeway flow:

\[ E_T = 1.5, \quad f_p = 1.00 \]

\[ f_{HV} = \frac{1}{1 + p_T(E_T - 1)} = \frac{1}{1 + 0.04(1.5 - 1)} = 0.980 \]

\[ v_i = \frac{v_i}{\text{PHF}_{fHV} f_p} = \frac{5.800}{(0.90)(0.980)(1.00)} = 6.576 \]

Onramp flow:

\[ f_{HV} = \frac{1}{1 + p_T(E_T - 1)} = \frac{1}{1 + 0.05(1.5 - 1)} = 0.976 \]

\[ v_i = \frac{V_i}{\text{PHF}_{fHV} f_p} = \frac{400}{(0.90)(0.976)(1.00)} = 455 \]

Offramp flow:

\[ v_i = \frac{V_i}{\text{PHF}_{fHV} f_p} = \frac{600}{(0.90)(0.976)(1.00)} = 683 \]

2. Merge:

Determine \( P_{FM} \) (Equation 4):

\[ P_{FM} = 0.2178 - 0.000125v_R + \frac{0.05887L_A}{\text{SFR}} \]

\[ = 0.2178 - 0.000125(455) + \frac{(0.05887)(250)}{70} = 0.3712 \]

Determine \( v_{12} \):

\[ v_{12} = v_f \times P_{FM} = (6.576)(0.3712) = 2.441 \]

Determine \( v_{FO} \) and \( v_{R12} \):

\[ v_{FO} = v_f + v_R = 6.576 + 455 = 7.031 \]

\[ v_{R12} = v_R + v_{12} = 455 + 2.441 = 2.896 \]

Check capacity (Table C.15):

7.031 < 9.200 under capacity

Check maximum flow in influence area:

2.896 < 4.600 OK

Estimate density:

\[ D_R = 3.402 + 0.00456v_R + 0.0048v_{12} - 0.01278L_A \]

\[ = 3.402 + 0.00456(455) + 0.0048(2.441) - 0.01278(250) = 14.00 \]

Determine level of service (Table C.18):

12 < 14.00 < 17 level of service C
3. Diverge:

\[ v_F = 6,576 + 455 = 7,031 \]

Determine \( P_{FD} \) (Equation 8):

\[ P_{FD} = 0.436 \]

Determine \( v_{12} \):

\[ v_{12} = v_R + (v_F - v_R)P_{FD} = 683 + (7,031 - 683)(0.436) = 3,451 \]

Check capacity (Tables C.16 and C.17):

7,031 < 9,200 under capacity
683 < 2,100 under capacity

Check maximum flow in influence area:

3,451 < 4,400 OK

Estimate density:

\[ D_R = 2.642 + 0.0053v_{12} - 0.0183L_D \]

\[ = 2.642 + 0.0053(3,451) - 0.0183(175) = 17.73 \]

Determine level of service:

17 < 17.73 < 22 level of service D

Diverge governs. Overall level of service is D

### 10.3.3 Two-Lane Highways

Chapter 20 of the *Highway Capacity Manual* discusses capacity and level of service for two-lane highways. Many of the procedures in this chapter are similar to those used for basic freeway sections, except that procedures for two-lane highways are influenced by the need for vehicles to pass in the face of oncoming traffic.

Figure 10.7 illustrates the methodology for analyzing two-lane highways. The HCM presents two procedures for analyzing two-lane highways, one applying to two-way segments and the other to directional segments. The procedure for analyzing two-way segments applies only to roads in level or rolling terrain. Two-lane highways in mountainous terrain and those containing grades of 3.0 percent or more with lengths 1.0 km or more should be analyzed as directional segments. Only the method for analyzing two-way segments is covered here; for information on the method for directional segments, see the *Highway Capacity Manual* itself.

The HCM gives the capacity of two-lane highways as 1,700 pc/h for each direction of travel and states that this capacity is nearly independent of the directional distribution of traffic, except that capacity will normally not exceed 3,200 pc/h for both directions of travel combined over extended lengths. Levels of service are described in Tables C.19 and C.20. For Class I highways (highways that serve a high percentage of long trips), levels of service depend on the percent time spent following in platoons.
and the average highway speed. For Class II highways, level of service depends only on the percent time spent following.

As in the case of basic freeway sections, the first step in calculating the level of service is determining the free-flow speed. Where actual speed measurements are not available, the free-flow speed may be estimated by

$$\text{FFS} = \text{BFFS} - f_{LS} - f_A$$  \hspace{1cm} (10.24)

where $f_{LS} =$ adjustment for lane and shoulder width, from Table C.21

$f_A =$ adjustment for access points, from Table C.22
The peak 15-min flow rate is given by

\[ v_p = \frac{V}{\text{PHF} f_G f_H} \]  

(10.25)

where \( f_G \) is a grade adjustment factor. Two different types of grade adjustment factors are used, depending on whether the analysis is intended to determine speeds or percent time spent following. Grade adjustment factors to be used to determine speeds are given in Table C.23, and those to be used in determining percent time spent following are given in Table C.24. Passenger car equivalents for determining \( f_H \) also depend on whether speed or percent time spent following is being calculated; those to be used in determining speed are given in Table C.25 and those to be used in determining percent time spent following are given in Table C.26.

Both \( f_G \) and the passenger car equivalents \( E_T \) and \( E_R \) depend on the flow rate \( v_p \); consequently, iterative calculations are required to find \( v_p \). The procedure for calculating \( v_p \) is to begin by setting \( v_p \) equal to \( V/\text{PHF} \). Then, using the appropriate values of \( f_G, E_T, \) and \( E_R \), calculate a new value for \( v_p \). If this is outside the flow limits for which \( f_G, E_T, \) and \( E_R \) were calculated, recalculate \( v_p \) with new values of \( f_G, E_T, \) and \( E_R \). Continue this process until the value of \( v_p \) is consistent with the flow ranges assumed in choosing \( f_G, E_T, \) and \( E_R \).

Once \( v_p \) is determined, average travel speed is estimated by

\[ \text{ATS} = \text{FFS} - 0.0125v_p - f_{np} \]  

(10.26)

where \( \text{ATS} \) = average travel speed for both directions of travel combined, km/h
\( f_{np} \) = adjustment for percentage of no-passing zones, from Table C.27

Percent time spent following is determined by

\[ \text{PTSF} = \text{BPTSF} + f_{dnp} \]  

(10.27)

where \( \text{PTSF} \) = percent time spent following
\( \text{BPTSF} \) = base percent time spent following
\( f_{dnp} \) = adjustment for combined effect of the directional distribution of traffic and percent no-passing zones, from Table C.28

The base percent time spent following, in turn, is given by

\[ \text{BPTSF} = 100 \left(1 - e^{(-0.000879v_p)}\right) \]  

(10.28)

The level of service is determined by first comparing \( v_p \) with the two-way capacity of 3,200 pc/h. If \( v_p \) is greater than capacity, the level of service is F. Furthermore, if the demand flow rate in either direction \( (v_p \times \text{directional split}) \) is greater than 1,700 pc/h, the level of service is F. For a segment on a Class I facility with demand less than capacity, the level of service is determined by comparing the speed and percent time spent following with the limits given in Table C.19, with the more restrictive case governing. For a segment on a Class II facility with demand less than capacity, the
level of service is determined by comparing the percent time spent following with the limits given in Table C.20.

**EXAMPLE PROBLEM 10.6** A Class I two-lane highway has a base free-flow speed of 100 km/h. Lane width is 3.6 m and shoulder width is 1.2 m. There are six access points per kilometer. The roadway is located in rolling terrain with 40 percent no-passing zones. The two-way traffic volume is 800 veh/h, with a PHF of 0.90. The directional split is 60/40. Traffic includes 5 percent trucks and 10 percent recreational vehicles. Determine the level of service.

Determine free-flow speed:

\[
\text{FFS} = \text{BFFS} - f_L - f_A
\]

\[
\text{BFFS} = 100 \text{ km/h (given)}
\]

\[
f_L = 2.1 \text{ (Table C.21)}
\]

\[
f_A = 2.5 \text{ (Table C.22)}
\]

\[
\text{FFS} = 100 - 2.1 - 2.5 = 95.4 \text{ km/h}
\]

Determine trial value of \(v_p\):

\[
v_p = \frac{V}{\text{PHF}} = \frac{800}{0.90} = 889 \text{ pc/h}
\]

Trial value of \(v_p\) is between 600 and 1,200

Determine \(v_p\) for speed calculation:

\[
v_p = \frac{V}{\text{PHF} f_G f_{HV}}
\]

\[
f_G = 0.93
\]

\[
E_T = 1.9
\]

\[
E_R = 1.1
\]

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

\[
= \frac{1}{1 + 0.05(1.9 - 1.0) + 0.10(1.1 - 1.0)} = 0.948
\]

\[
v_p = \frac{800}{(0.90)(0.93)(0.948)} = 1,008
\]

\[600 < 1,008 < 1,200 \quad \text{OK}\]

Check capacity:

\[1,008 < 3,200 \quad \text{OK}\]

\[(0.60)(1,008) = 605 < 1,700 \quad \text{OK}\]
Determine $v_p$ for percent time spent following:

$$v_p = \frac{V}{\text{PHF} f_G^c f_{HV}}$$

$f_G = 0.94$

$E_T = 1.5$

$E_R = 1.0$

$$f_{HV} = \frac{1}{1 + P_T (E_T - 1) + P_R (E_R - 1)} = 0.976$$

$$v_p = \frac{800}{(0.90)(0.94)(0.976)} = 969$$

$600 < 969 < 1,200 \quad \text{OK}$

Determine average travel speed:

$$\text{ATS} = \text{FFS} - 0.0125v_p - f_{dp}$$

$$f_{dp} = 2.5 - \frac{1,008 - 1,000}{1,200 - 1,000} (2.5 - 2.0) = 2.5 \quad (\text{Table C.27, interpolated})$$

$$\text{ATS} = 95.4 - 0.0125(1,008) - 2.5 = 80.3 \text{ km/h}$$

$80 < 80.3 < 90 \quad \text{level of service B}$

Determine percent time spent following:

$$\text{BPTSF} = 100(e^{-0.000879v_p}) = 100(e^{-0.000879(969)}) = 42.7$$

$$\text{PTSF} = \text{BPTSF} + f_{dp}$$

$$f_{dp} = 10.3 - \frac{969 - 800}{1,400 - 800}(10.3 - 5.4) = 8.9 \quad (\text{Table C.28, interpolated})$$

$$\text{PTSF} = 42.7 + 8.9 = 51.6 \text{ percent}$$

$50 < 51.6 < 65 \quad \text{level of service C}$

Percent time spent following governs. *Level of service for segment is C*

### 10.3.4 Signalized Intersections

Chapter 16 of the *Highway Capacity Manual* deals with the capacity of signalized intersections. It is not really meaningful to speak of capacities and service flow rates for intersections as a whole, unless data about the efficiency of signal timing and coordination (discussed in Chapter 11 of this book) are available. Also, because of the
importance of signal timing and coordination in determining the relationship between flow rates and delay at signals, the relationship between intersection capacity and level of service (defined in the HCM in terms of delay per vehicle) is by no means straightforward. As a result, the procedures for analysis of signalized intersections are very detailed and require a large amount of data.

Because of the complexity of the method for signalized intersections, the HCM also offers a simplified quick estimation procedure (Appendix A to Chapter 10 of the HCM) that can be used where only minimal data are available and only approximate results are desired. This method makes use of a number of assumptions and default values that are documented in Chapter 10 of the HCM. It also involves use of a method for determining an estimated signal timing that is similar to the “Highway Capacity Manual method” described in Section 11.2.

The method presented in Chapter 16 of the HCM is quite complicated and requires very detailed traffic, geometric, and environmental information about the intersection to be analyzed. As a consequence, it can ordinarily be applied only to existing intersections, for which such detailed information is available. Because of the complexity of this method, only an overview will be presented here, with emphasis on the calculation of saturation flows and control delay. For further details, see the Highway Capacity Manual itself.

The signalized intersection method involves five basic steps:

1. Determine input parameters
2. Determine lane grouping and demand flow rates
3. Determine saturation flow rate
4. Determine capacity and volume to capacity ratios
5. Determine performance measures

Figure 10.8 is a flow chart illustrating the method.

Input parameters include data on intersection geometry, traffic volumes and conditions, and signalization. Lane grouping and determination of demand flow rates involves establishment of lane groups with more-or-less homogeneous traffic flow conditions, conversion of hourly volumes to equivalent peak 15-min flow rates, and adjustment for right turns on red, if applicable. Determination of saturation flow involves correction of a basic saturation flow rate per lane by means of a series of adjustment factors. Determination of capacity and volume/capacity ratios involves manipulation of volumes and saturation flow rates to compute capacities and v/c ratios for each lane group. Determination of performance measures involves estimation of delay for each lane group and for the intersection as a whole, determination of levels of service based on control delay per vehicle, estimation of queue lengths.

Saturation flows are estimated by establishing a base saturation flow rate and then adjusting it to account for a variety of prevailing conditions. These include lane width, proportion of heavy vehicles in the traffic stream, approach grade, existence of a parking lane adjacent to the lane group, blockage by vehicles parking and unparking, blockage by transit buses, area type, lane use, right and left turns in the lane group, and the effects of interference by pedestrians and bicycles on right and left turn operation. Base saturation flow $S_0$ is considered to be 1,900 passenger cars per hour of green times the number of lanes. Adjustment factors have been determined by regression
analysis. The resulting regression equations are as follows:

\[ f_w = 1 + \frac{W - 3.6}{9} \]  

where \( f_w \) is the lane width adjustment factor and \( W \) is lane width in meters.

\[ f_{HV} = \frac{100}{100 + \%HV(E_T - 1)} \]  

where \( f_{HV} \) is the heavy vehicle factor, \%HV is the fraction of heavy vehicles and \( E_T \) is the number of passenger car equivalents for each heavy vehicle. In this application, \( E_T \) is always taken to be 2, so the heavy vehicle factor is more simply

\[ f_{HV} = \frac{100}{100 + \%HV} \]  

The grade factor \( f_g \) is given by

\[ f_g = 1 - \frac{\%G}{200} \]
where \( \%G \) is the grade in percent. The *parking factor* \( f_p \) is given by

\[
f_p = \frac{N - 0.1 - 18N_m/3,600}{N}
\]

(10.33)

where \( N \) is the number of lanes and \( N_m \) is the number of parking maneuvers per hour. The *adjustment factor for bus blockage* \( f_{bb} \) is given by

\[
f_{bb} = \frac{N - 14.4N_b}{3,600}
\]

(10.34)

where \( N \) is the number of lanes and \( N_b \) is the number of buses stopping per hour. The *adjustment factor for area type* \( f_a \) is 0.900 for central business districts (CBDs) and 1.000 for all other areas. The lane utilization factor \( f_{LU} \) is given by

\[
f_{LU} = \frac{v_g}{v_{g1}N}
\]

(10.35)

where \( v_g \) is the unadjusted flow rate for the lane group, \( v_{g1} \) is the unadjusted flow in the lane with the highest volume, and \( N \) is the number of lanes in the lane group. The *HCM* recommends that lane utilization be observed in the field, but where such observations are not available, the values in Table C.29 may be used. The *adjustment factor for right turns* \( f_{RT} \), is 0.85 exclusive right turn lanes,

\[
f_{RT} = 1.0 - (0.15)P_{RT}
\]

(10.36)

for shared lanes, and

\[
f_{RT} = 1.0 - (0.135)P_{RT}
\]

(10.37)

for single-lane approaches, where \( P_{RT} \) is the proportion of right turns in the lane group. The adjustment factor for left turns, \( f_{LT} \) is 0.95 for exclusive left turns with protected phasing and

\[
f_{LT} = \frac{1}{1.0 + 0.05P_{LT}}
\]

(10.38)

for other cases with protected phasing. A protected phase is a period of time during which a particular movement has the green, and there are no conflicting movements. Other cases are rather complicated, and will not be covered here. See the *HCM* for details. The *adjustments for pedestrian/bicycle blockage for left and right turns* \( f_{Lpb} \) and \( f_{Rpb} \) are calculated by a complicated procedure that will also not be covered here.

**EXAMPLE PROBLEM 10.7** The diagram below shows a lane group at an intersection. The intersection is located in an outlying business district. Traffic includes 6 percent heavy vehicles. The grade approaching the intersection is \(-1.0\%\). There are 15 parking maneuvers per hour and 4 bus blockages per hour. Right turns amount to 15 percent of the total flow; the adjustment for pedestrian/bicycle blockage for right turns is 0.993. There are no
left turns, due to the adjacent left turn lane. Find the saturation flow rate for this lane group.

Base saturation flow:

\[ S_0 = 1,900 \]

Number of lanes:

\[ N = 2 \]

Factors:

\[ f_w = 1 + \frac{W - 3.6}{9} = 1 + \frac{3.6 - 3.6}{9} = 1.00 \]

\[ f_{LV} = \frac{100}{100 + \%HV} = \frac{100}{100 + 6} = 0.943 \]

\[ f_g = 1 - \frac{\%G}{200} = 1 - \frac{1}{200} = 1.005 \]

\[ f_p = \frac{N - 0.1 - 18N_m/3,600}{2} = 2 - 0.1 - (18)(15)/3,600 = 0.9125 \]

\[ f_{bb} = \frac{N - 14.4N_m/3,600}{2} = 2 - (14.4)(4)/3,600 = 0.992 \]

\[ f_a = 1.000 \text{ (non-CBD)} \]

\[ f_{LU} = 0.952 \text{ (default value, Table C.29)} \]

\[ f_{RT} = 1.0 - 0.15P_{RT} = 1.0 - 0.15(0.15) = 0.978 \]

\[ f_{LT} = 1.00 \text{ (not applicable)} \]

\[ f_{Rpb} = 0.993 \text{ (given)} \]

\[ f_{Lpb} = 1.00 \text{ (not applicable)} \]

Calculate saturation flow rate:

\[ s = s_0 N f_w f_{LV} f_g f_p f_{bb} f_a f_{LU} f_{RT} f_{LT} f_{Rpb} f_{Lpb} \]

\[ s = 1,900(2)(1.000)(0.943)(1.005)(0.9125)(0.992)(1.000) \]

\[ \times (0.952)(0.978)(1.000)(0.993)(1.000) = 3,014 \text{ veh/h} \]

Control delays are estimated by

\[ d = d_1PF + d_2 + d_3 \quad (10.39) \]
where \( d \) = control delay per vehicle, s/veh
\( d_1 \) = uniform control delay assuming uniform arrivals, s/veh
\( PF \) = progression adjustment factor accounting for effects of signal progression (see Section 11.3 for information on signal progression)
\( d_2 \) = incremental delay to account for random arrivals and oversaturated queues
\( d_3 \) = initial queue delay to account for delay due to any initial queue at the beginning of the analysis period

Uniform delay is given by

\[
d_1 = \frac{0.50C(1 - g/C)^2}{1 - \left[ \text{Min}(1, X)(g/C) \right]}
\]

(10.40)

where \( C \) = cycle length
\( g \) = effective green time for lane group, s
\( X = v/c \) ratio or degree of saturation for lane group

Incremental delay is given by

\[
d_2 = 900T \left[ (X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}} \right]
\]

(10.41)

where \( T \) = duration of analysis period, h
\( k \) = incremental delay factor that is dependent on controller settings
\( I \) = upstream filtering/metering adjustment factor
\( c \) = lane group capacity in veh/h

The simplest case (and the only one considered here) is when the signal is isolated (that is, it is not part of a signal progression), the green is pretimed, and there is no initial queue. In that case, arrivals are assumed to be random and the values of \( PF, k, \) and \( I \) are \( PF = 1.000, k = 0.50, \) and \( I = 1.000, \) and \( d_3 = 0. \) The relationship between delay and level of service is given in Table C.30.

**EXAMPLE PROBLEM 10.8** An intersection approach at an isolated pretimed signal with a cycle length of 80 s has a saturation flow rate of 3,000 veh/h. The length of the green is 24 s. The \( v/c \) ratio is 0.90. What is the level of service, if control delay is measured over a 15 min interval?

\[
d = d_1PF + d_2 + d_3
\]

Determine uniform delay:

\[
d_1 = \frac{0.50C(1 - g/C)^2}{1 - \left[ \text{Min}(1, X)(g/C) \right]}
\]

\( g/C = 24/80 = 0.30 \)

\[
d_1 = \frac{0.50(80)(1 - 0.30)^2}{1 - \left[ \text{Min}(1, 1.000)(0.30) \right]} = \frac{0.50(80)(1 - 0.30)^2}{1 - [(0.90)(0.30)]} = 26.8 \text{ s}
\]

Determine incremental delay:

\[
d_2 = 900T \left[ (X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}} \right]
\]
282 CHAPTER 10: Capacity and Level of Service

\[
k = 0.50 \text{ (pretimed signal)}
\]

\[
l = 1.000 \text{ (isolated intersection)}
\]

\[
d_2 = 900(0.25) \left[ (0.90 - 1) + \sqrt{(0.90 - 1)^2 + \frac{8(0.50)(1.000)(0.90)}{(3,000)(0.25)}} \right] = 4.9 \text{ s}
\]

Determine initial queue delay:

\[
d_1 = 0.0 \text{ (no initial queue)}
\]

Determine total delay and level of service:

\[
PF = 1.000 \text{ (isolated signal, random arrivals)}
\]

\[
d = (26.8)(1.000) + 4.9 + 0.0 = 31.7 \text{ s}
\]

\[
20 < 31.7 < 35 \quad \text{level of service C}
\]

10.4 SUMMARY

Capacity is defined as the maximum number of vehicles, passengers, or the like, per unit time, which can be accommodated by a given facility or system under given conditions, with a reasonable expectation of occurrence. The concept of service volume is closely related, and is defined as the maximum number of vehicles, passengers, or the like, that can be accommodated by a given facility or system under given conditions at a given level of service. Level of service criteria are related to the quality of traffic flow, and include such things as travel times or speeds, total delay, probability of delay, safety, and comfort. Capacities of airports and rail lines are heavily dependent on the details of their control systems. Since control systems may differ on a case-by-case basis, capacities are calculated by means of analytical formulas, space–time diagrams, and simulations. Highway capacities are heavily dependent on driver behavior, and are calculated by using flow models based on empirical data. The Highway Capacity Manual is the standard North American reference on highway capacity. It contains methods for calculating capacities and service volumes for a variety of facilities. Highway Capacity Manual methods for calculating capacities of basic freeway segments, ramps and ramp junctions, two-lane rural highways, and signalized intersections have been discussed in this chapter.

REFERENCES

PROBLEMS

10.1 An airport runway handling arrivals only is used by two classes of aircraft, a conventional jet with an approach speed of 120 nautical miles per hour (NM/h) and a heavy jet with an approach speed of 150 NM/h. The common approach path is 6 NM long; 60 percent of the aircraft are conventional jets and 40 percent are heavy jets. Buffer times in seconds are as given in the table below. Determine the capacity of the runway in aircraft per hour.

<table>
<thead>
<tr>
<th>Trailing aircraft speed</th>
<th>Lead aircraft speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>25</td>
</tr>
<tr>
<td>150</td>
<td>30</td>
</tr>
</tbody>
</table>

10.2 An airport runway handling arrivals only is used by two classes of aircraft, a conventional jet with an approach speed of 150 NM/h and a conventional jet with an approach speed of 180 NM/h. The common approach path is 6 NM long; 40 percent of the aircraft travel at 150 knots and 60 percent at 180 knots. Buffer times in seconds are as given in the table below. Determine the capacity of the runway in aircraft per hour.

<table>
<thead>
<tr>
<th>Trailing aircraft speed</th>
<th>Lead aircraft speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>150</td>
<td>33</td>
</tr>
<tr>
<td>180</td>
<td>33</td>
</tr>
</tbody>
</table>

10.3 An airport runway handling arrivals only is used by two classes of aircraft, a conventional jet with an approach speed of 120 NM/h and a heavy jet with an approach speed of 180 NM/h. The common approach path is 6 NM long; 70 percent of the aircraft are conventional jets and 30 percent are heavy jets. Buffer times in seconds are as given in the table below. Determine the capacity of the runway in aircraft per hour.

<table>
<thead>
<tr>
<th>Trailing aircraft speed</th>
<th>Lead aircraft speed</th>
</tr>
</thead>
<tbody>
<tr>
<td>120</td>
<td>30</td>
</tr>
<tr>
<td>180</td>
<td>35</td>
</tr>
</tbody>
</table>

10.4 A freeway in a mountainous recreation area has a 4 percent upgrade 2.4 km long. Base free-flow speed is 120 km/h. The traffic stream includes 15 percent recreational vehicles and 6 percent trucks and buses. There are two 3.6 m lanes in each direction and no lateral obstructions. Interchanges are more than 3.5 km apart. PHF is 0.87. Based on past experience, it is determined that the adjustment factor for the character of the traffic stream should be 0.85. What is the maximum hourly volume that can be accommodated at level of service C?

10.5 A rural freeway has two 3.6 m lanes in each direction and a traffic stream composed of 12 percent recreational vehicles and 8 percent trucks and buses. There is a 3 percent upgrade, 1.7 km in length. There are no lateral obstructions. Interchanges are approximately 5 km apart. Base free-flow speed is 120 km/h. If the current maximum hourly
volume is 1,790 veh/h, with a PHF of 0.90, and the adjustment factor for the character of the traffic stream is 0.90, what is the level of service?

10.6 An urban freeway has four 3.6 m lanes in one direction and a traffic stream composed of 10 percent trucks and buses (recreational vehicles are negligible). PHF is 0.92. There are no lateral obstructions. Base free-flow speed is 110 km/h. Interchanges are spaced about 1.6 km apart. What is the maximum hourly volume at level of service D on a 3 percent upgrade, 2 km long?

10.7 An urban freeway presently has three 3.6 m lanes on a 3 percent upgrade 2.8 km long. The traffic includes 8 percent trucks and buses (recreational vehicles are negligible). There are no lateral obstructions. Interchanges are about 1.3 km apart. PHF is 0.90. Base free-flow speed is 110 km/h.
   (a) What maximum hourly volume can currently be accommodated by the upgrade (at capacity)?
   (b) How much could this be increased by widening and remarking the existing roadway to provide four 3-m lanes up the grade?

10.8 A rural freeway has a 5 percent upgrade 1.2 km long. Expected traffic composition is 10 percent trucks and buses and 10 percent recreational vehicles. The adjustment factor for the character of the traffic stream is expected to be 0.75. Base free-flow speed is 120 km/h. Interchanges are about 8 km apart. If the hourly volume is expected to be 1,950 VPH with a PHF of 0.85, how many lanes are needed to provide level of service C?

10.9 An urban freeway has three 3.6 m lanes in each direction. There are no lateral obstructions. Base free-flow speed is 110 km/h. Interchanges are about 1.1 km apart. The traffic stream includes 12 percent trucks and buses and 2 percent recreational vehicles. If the present peak hour volume is 2,200 VPH with a PHF of 0.92, what is the level of service on a 3 percent grade, 1.9 km long?

10.10 What is the level of service for the ramp combination and traffic conditions shown below? PHF is 0.85. The terrain is level. Traffic is urban commute traffic. Free-flow speed on the freeway is 110 km/h and free-flow speed on the ramps is 70 km/h.

10.11 What is the level of service for the ramp combination and traffic conditions shown below? PHF is 0.92. The terrain is level. Free-flow speed on the freeway is 100 km/h and free-flow speed on the ramps is 60 km/h.
10.12 What is the level of service for the ramp combination and traffic conditions shown below? PHF is 0.78. The terrain is level. Traffic is urban commute traffic. Free-flow speed on the freeway is 110 km/h and free-flow speed on the ramps is 70 km/h.

10.13 What is the level of service for the ramp combination and traffic conditions shown below? PHF is 0.87. The terrain is level. Free-flow speed on the freeway is 110 km/h and free-flow speed on the ramps is 70 km/h.

10.14 What is the level of service for the ramp combination and traffic conditions shown below? PHF is 0.81. The terrain is level. Free-flow speed on the freeway is 100 km/h and free-flow speed on the ramps is 70 km/h.

10.15 What is the level of service for the ramp combination and traffic conditions shown below? PHF is 0.86. The terrain is level. Free-flow speed on the freeway is 110 km/h and free-flow speed on the ramps is 60 km/h.

10.16 A Class I two-lane highway has a base free-flow speed of 110 km/h. Lane width is 3.6 m and shoulder width is 2.4 m. There are six access points per kilometer. The roadway is located in rolling terrain with 40 percent no-passing zones. The two-way traffic volume
is 1000 veh/h, with a PHF of 0.90. The directional split is 70/30. Traffic includes 10 percent trucks and 5 percent recreational vehicles. Determine the level of service.

10.17 A Class I two-lane highway has a base free-flow speed of 100 km/h. Lane width is 3.3 m and shoulder width is 1.2 m. There are 12 access points per kilometer. The roadway is located in rolling terrain with 60 percent no-passing zones. The two-way traffic volume is 700 veh/h, with a PHF of 0.90. The directional split is 60/40. Traffic includes 8 percent trucks and 12 percent recreational vehicles. Determine the level of service.

10.18 A Class I two-lane highway has a base free-flow speed of 90 km/h. Lane width is 3.6 m and shoulder width is 1.7 m. There are 18 access points per kilometer. The roadway is located in level terrain with 20 percent no-passing zones. The two-way traffic volume is 1,200 veh/h, with a PHF of 0.90. The directional split is 50/50. Traffic includes 12 percent trucks and 4 percent recreational vehicles. Determine the level of service.

10.19 Determine the saturation flow for the lane group below (the eastbound dual left turn lane), given the information below:

- Percent heavy vehicles: 6
- Grade: +2%
- Area type: non-CBD
- $f_{Lpb}$: 0.95

10.20 Determine the saturation flow rate for the lane group shown below (the northbound one-way approach) given the information below. Flow in the left lane is 56 percent of the total flow for the approach.

- Percent heavy vehicles: 6
- Grade: -2%
- Parking maneuvers: 20/h
- Bus blockage: 10/h
- Area type: CBD
- Conflicting pedestrians: 200/h
- RT, % of total flow: 15
- LT, % of total flow: 10
- $f_{Lpb}$: 0.995
- $f_{Rpb}$: 0.978
Problems 287

10.21 Determine the saturation flow rate for the lane group shown below (the eastbound through and right turn) given the following information:

- Percent heavy vehicles: 8
- Grade: +1%
- Parking maneuvers: 10/h
- Bus blockage: 10/h
- Area type: non-CBD
- Conflicting pedestrians: 50/h
- RT, % of total flow: 20
- \( f_{Rpb} \): 0.995

10.22 An intersection approach at an isolated pretimed signal with a cycle length of 75 s has a saturation flow rate of 3,300 veh/h. The length of the green is 25 s. The \( v/c \) ratio is 0.92. What is the level of service, if control delay is measured over a 15 min interval?

10.23 An intersection approach at an isolated pretimed signal with a cycle length of 60 s has a saturation flow rate of 2,800 veh/h. The length of the green is 18 s. The \( v/c \) ratio is 0.95. What is the level of service, if control delay is measured over a 15 min interval?
10.24 An intersection approach at an isolated pretimed signal with a cycle length of 90 s has a saturation flow rate of 1,700 veh/h. The length of the green is 25 s. The v/c ratio is 0.93. What is the level of service, if control delay is measured over a 15 min interval?

COMPUTER EXERCISES

10.1 Programming. Write a computer program to calculate the capacity (in aircraft/h) of a runway used for arrivals only. The program should be able to handle up to five different classes of aircraft. The minimum distance separation matrix, percentage of each class of aircraft, buffer matrix, approach speeds, and length of the common approach path are to be provided by the user as input. Output should include a matrix of minimum distance separations, a matrix of unbuffered time separations, a matrix of buffered time separations, the weighted average time separation, and the capacity. Test the program using the data from Problem 10.3 and the following data:

<table>
<thead>
<tr>
<th>Aircraft class</th>
<th>Type</th>
<th>Speed, knots</th>
<th>Percent of traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Heavy</td>
<td>150</td>
<td>40</td>
</tr>
<tr>
<td>2</td>
<td>Conventional</td>
<td>150</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>Conventional</td>
<td>135</td>
<td>5</td>
</tr>
<tr>
<td>4</td>
<td>Conventional</td>
<td>120</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>Heavy</td>
<td>180</td>
<td>15</td>
</tr>
</tbody>
</table>

Buffer matrix:

<table>
<thead>
<tr>
<th>Trailing aircraft class</th>
<th>Lead aircraft class</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>23</td>
<td>23</td>
<td>20</td>
<td>18</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>23</td>
<td>23</td>
<td>20</td>
<td>18</td>
<td>27</td>
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</tr>
<tr>
<td>3</td>
<td>20</td>
<td>20</td>
<td>18</td>
<td>16</td>
<td>24</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>18</td>
<td>16</td>
<td>15</td>
<td>22</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>27</td>
<td>27</td>
<td>24</td>
<td>22</td>
<td>32</td>
<td></td>
</tr>
</tbody>
</table>

10.2 Programming. Write a program to determine the level of service for an onramp followed by an offramp for a four-, six-, or eight-lane freeway. Input should be hourly volumes; acceleration and deceleration lane lengths; distance between the ramps; percentages of trucks; passenger car equivalents for trucks (based on generalized terrain); PHF for the onramp, offramp, and freeway upstream of the onramp; driver population factor; and freeflow speeds for the freeway, onramp, and offramp. Output should be $V_{12}$ for merge, $V_{12}$ for diverge, density in lanes 1 and 2 for both merge and diverge (if applicable), freeway flow between the ramps and the level of service for the merge, diverge, and the ramp combination as a whole. If the flow in the influence area exceeds the maximum desirable level for either the merge or the diverge, the program should write out a warning message to this effect. All output flows should be 15 min flow rates in pc/h. Test the program using the data from Problems 10.11 and 10.12.

10.3 Spreadsheet. Use a spreadsheet to calculate the capacity of a runway used for arrivals only. The spreadsheet should document the speed and fraction of traffic for each aircraft type and should present in matrix form the time separation, time buffer, and fraction of
traffic for each aircraft pair. The spread sheet should also calculate the average time separation is seconds per aircraft and the runway capacity in aircraft per hour. Use the spread sheet to work one or more of Problems 10.1, 10.2, and 10.3.

10.4 Spread Sheet. Use a spread sheet to work one or more of Problems 10.5, 10.6, and 10.9. The spread sheet should document all input data and all correction factors (which the user should look up from the appropriate tables), and should calculate the maximum hourly volume, 15 min flow rate, or density, as appropriate, and document the level of service.

10.5 Spread Sheet. Design and use a spread sheet to calculate the level of service for combinations of on- and offramp junctions. The spread sheet should document all input data and correction factors and be used to calculate all conversions of volumes to peak flow rates in pc/h. It should also document the equation or equations used to calculate $P_{FM}$ and/or, $P_{FD}$, the calculation of all flows used in the capacity checks, the capacity checks themselves, the density calculation, and the level of service. Use the spread sheet to solve one or more of Problems 10.10 to 10.15.

10.6 Spread Sheet. Use a spread sheet to calculate the level of service for a two-lane highway. The spread sheet should document all input data and correction factors and calculate the free-flow speed, 15 min flow rate in pc/h, average travel speed, and percent time spent following. The spread sheet should also document the level of service. Use the spread sheet to solve one or more of Problems 10.16, 10.17, and 10.18.

10.7 Spread Sheet. Use a spread sheet to calculate the saturation flow rate for a signalized intersection approach involving either no left turns or protected left turns and either no right turns or permitted right turns without separate lanes. The spread sheet should be used to document all input data, to calculate correction factors using equations where appropriate, and to calculate and document the saturation flow. Use the spread sheet to solve Problem 10.21.

10.8 Spread Sheet. Use a spread sheet to determine the level of service of a lane group at an intersection with an isolated pretimed signal. The spread sheet should document the uniform delay, the incremental delay, and the level of service. Use the spread sheet to solve one or more of Problems 10.22, 10.23, and 10.24.

DESIGN EXERCISE

10.1 The layout, profile, and projected peak period traffic volumes for a new suburban freeway that is intended to fill in a gap in an existing freeway system are shown on page 290. Horizontal distances are given in 100 m stations. Although the state Transportation Department considers this to be a high priority project, it is in competition with other local projects for funding. An existing parallel facility 8 km away experiences significant peak period congestion. Although no specific projections are available, local experience suggests that about 2 percent trucks and buses and a similar percentage of recreational vehicles are likely to use the proposed freeway during peak periods. Design the number of lanes for each section of the proposed freeway. You may select any design standards you believe to be appropriate for this type of roadway. Defend your design decisions in a brief written report discussing design standards, design objectives and constraints, major design decisions, and your rationale for your decisions.