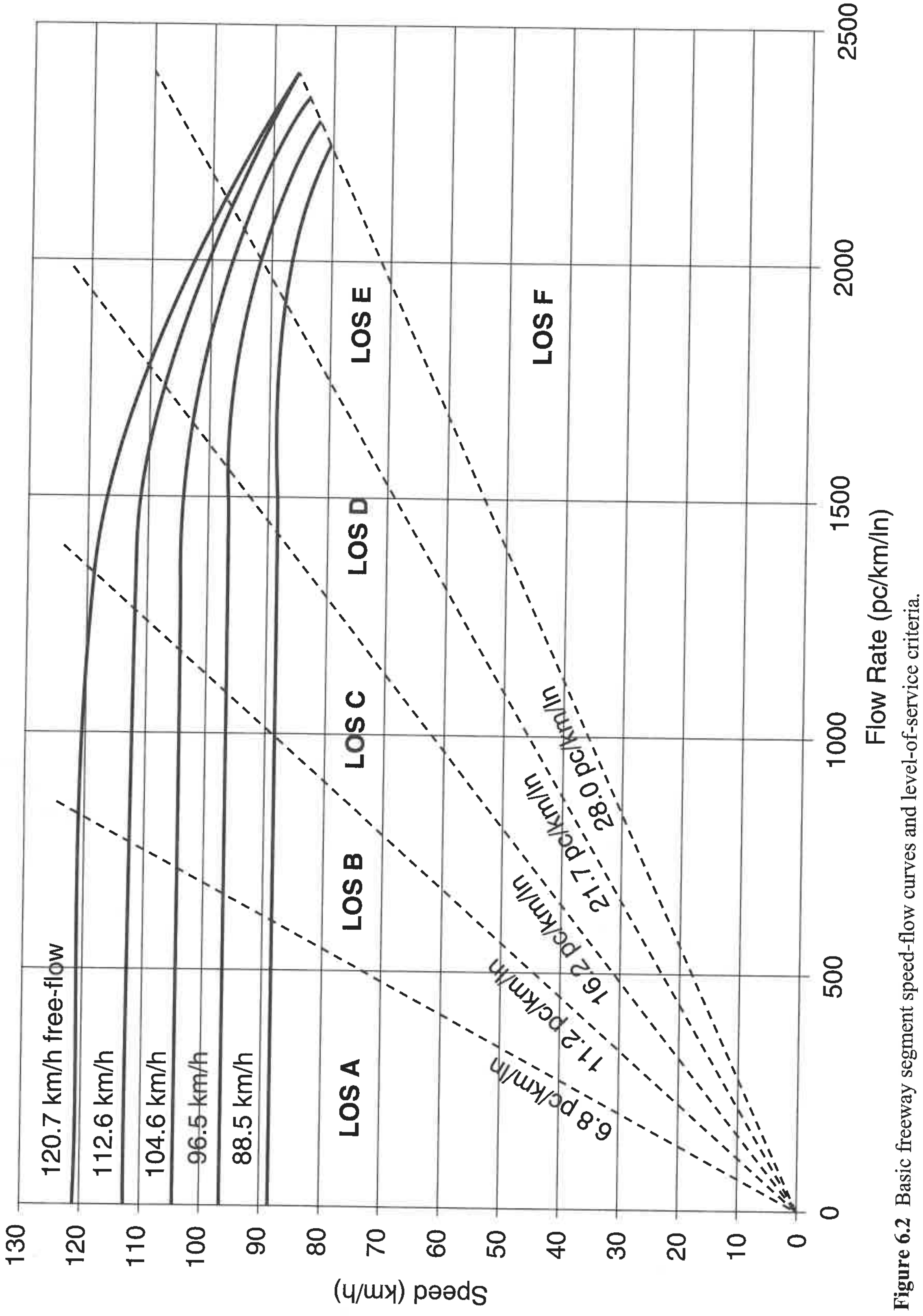

Figure 6.2 Basic freeway segment speed-flow curves and level-of-service criteria.

Adapted from Exhibit 11-6, p. 11-8 with permission of the Transportation Research Board, *Highway Capacity Manual* 2010, Copyright, National Academy of Sciences, Washington, D.C. Figure has been converted to SI units for use in this edition. Values from the source material are in U.S. Customary units.

Table 6.1 LOS Criteria for Basic Freeway Segments

Criterion	LOS				
	A	B	C	D	E
FFS = 120.7 km/h					
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	28.0
Average speed (km/h)	120.7	118.7	109.9	98.0	85.7
Maximum *v*/*c*	0.34	0.55	0.74	0.89	1.00
Maximum flow rate (pc/h/ln)	820	1330	1780	2125	2400
FFS = 112.6 km/h					
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	28.0
Average speed (km/h)	112.6	112.6	107.2	97.2	85.7
Maximum *v*/*c*	0.32	0.53	0.72	0.88	1.00
Maximum flow rate (pc/h/ln)	765	1260	1735	2110	2400
FFS = 104.6 km/h					
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	28.0
Average speed (km/h)	104.6	104.6	102.9	94.8	84.0
Maximum *v*/*c*	0.30	0.50	0.71	0.87	1.00
Maximum flow rate (pc/h/ln)	710	1170	1670	2055	2350
FFS = 96.5 km/h					
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	28.0
Average speed (km/h)	96.5	96.5	96.5	92.0	82.2
Maximum *v*/*c*	0.29	0.47	0.68	0.87	1.00
Maximum flow rate (pc/h/ln)	655	1080	1565	1995	2300
FFS = 88.5 km/h					
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	28.0
Average speed (km/h)	88.5	88.5	88.5	88.0	80.4
Maximum *v*/*c*	0.27	0.44	0.64	0.85	1.00
Maximum flow rate (pc/h/ln)	600	990	1435	1910	2250

Note: Density is the primary determinant of LOS. Maximum flow rate values are rounded to the nearest 5 passenger cars.

Table 6.2 Relationship Between Free-Flow Speed and Capacity on Basic Freeway Segments

Source: Transportation Research Board, *Highway Capacity Manual* 2010. Washington, D.C. National Academy of Sciences. Table has been converted to SI units for use in this edition. Values from the source material are in U.S. Customary units.

Free-flow speed (km/h)	Capacity (pc/h/ln)
120.7	2400
112.6	2400
104.6	2350
96.5	2300
88.5	2250

6.4.3 Determine Free-Flow Speed

For basic freeway segments, *FFS* is the mean speed of passenger cars operating in flow rates up to 1300 passenger cars per hour per lane (pc/h/ln). If *FFS* is to be estimated rather than measured, the following equation can be used. It accounts for the roadway characteristics of lane width, right-shoulder lateral clearance, and ramp density.

$$FFS = 121.3 - f_{LW} - f_{LC} - 7.96TRD^{0.84} \tag{6.2}$$

where

FFS = estimated free-flow speed in km/h,
f_{LW} = adjustment for lane width in km/h,
f_{LC} = adjustment for lateral clearance in km/h,
$7.96TRD^{0.84}$ = adjustment for total ramp density in km/h (with TRD in ramps/km).

The constant value of 121.3 in Eq. 6.2 is considered to be the base free-flow speed (*BFFS*) and applies to freeways in urban and rural areas. The HCM [Transportation Research Board 2010] recommends that the calculated free-flow speed be rounded to the nearest *FFS* values shown in Figure 6.2 and Table 6.1. The following sections describe the procedures for estimating the adjustment factor values.

Lane Width Adjustment

When lane widths are narrower than the base 3.6 m, the adjustment factor f_{LW} is used to reflect the impact on free-flow speed. Such an adjustment is needed because narrow lanes cause traffic to slow as a result of reduced psychological comfort and limits on driver maneuvering and accident avoidance options. Thus, *FFS* under these conditions is less than the value that would be observed if base lane widths were provided. The adjustment factors used in current practice are presented in Table 6.3.

Table 6.3 Adjustment for Lane Width

Lane width (m)	Reduction in free-flow speed, f_{LW} (km/h)
3.6	0.0
3.3	3.1
3.0	10.6

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Lateral Clearance Adjustment

When obstructions are closer than 1.8 m (at the roadside) from the traveled pavement, the adjustment factor f_{LC} is used to reflect the impact on *FFS*. Again, these conditions lead to reduced psychological comfort for the driver and consequently reduced speeds. An obstruction is a right-side object that can either be continuous (such as a retaining wall or barrier) or periodic (such as light posts or utility poles). Table 6.4 provides corrections for obstructions on the right side of the roadway.

Table 6.4 Adjustment for Right-Shoulder Lateral Clearance

Right-shoulder lateral clearance (m)	Reduction in free-flow speed, f_{LC} (km/h), lanes in one direction			
	2	3	4	≥5
≥ 1.8	0.0	0.0	0.0	0.0
1.5	1.0	0.7	0.3	0.2
1.2	1.9	1.3	0.7	0.4
0.9	2.9	1.9	1.0	0.6
0.6	3.9	2.6	1.3	0.8
0.3	4.8	3.2	1.6	1.1
0	5.8	3.9	1.9	1.3

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Total Ramp Density Adjustment

Ramp density provides a measure of the impact of merging and diverging traffic on free-flow speed. Total ramp density is the number of on- and off-ramps (in one direction) within a distance of 5 kilometers upstream and 5 kilometers downstream of the midpoint of the analysis segment, divided by 10 kilometers.

6.4.4 Determine Analysis Flow Rate

The analysis flow rate is calculated using the following equation:

$$v_p = \frac{V}{PHF \times N \times f_{HV} \times f_p} \tag{6.3}$$

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,
f_{HV} = heavy-vehicle adjustment factor, and
f_p = driver population factor.

The adjustment factors PHF, f_{HV}, and f_p are described next.

Peak-Hour Factor

As previously mentioned, vehicle arrivals during the period of analysis [typically the highest hourly volume within a 24-h period (peak hour)] will likely be nonuniform. To account for this varying arrival rate, the peak 15-min vehicle arrival rate within the analysis hour is usually used for practical traffic analysis purposes. The peak-hour factor has been developed for this purpose, and is defined as the ratio of the hourly volume to the maximum 15-min flow rate expanded to an hourly volume, as follows:

$$PHF = \frac{V}{V_{15} \times 4} \tag{6.4}$$

where

PHF = peak-hour factor,
V = hourly volume for hour of analysis,
V_{15} = maximum 15-min volume within hour of analysis, and
4 = number of 15-min periods per hour.

Equation 6.4 indicates that the further the *PHF* is from unity, the more *peaked* or nonuniform the traffic flow is during the hour. For example, consider two roads both of which have a peak-hour volume, *V*, of 1800 veh/h. The first road has 600 vehicles arriving in the highest 15-min interval, and the second road has 500 vehicles arriving in the highest 15-min interval. The first road has a more nonuniform flow, as indicated by its *PHF* of 0.75 [1800/(600 × 4)], which is further from unity than the second road's *PHF* of 0.90 [1800/(500 × 4)].

Heavy-Vehicle Adjustment

Large trucks, buses, and recreational vehicles have performance characteristics (slow acceleration and inferior braking) and dimensions (length, height, and width) that have an adverse effect on roadway capacity. Recall that base conditions stipulate that no heavy vehicles are present in the traffic stream, and when prevailing conditions indicate the presence of such vehicles, the adjustment factor f_{HV} is used to translate the traffic stream from base to prevailing conditions. The f_{HV} correction term is found using a two-step process. The first step is to determine the passenger car equivalent (PCE) for each large truck, bus, and recreational vehicle in the traffic stream. These values represent the number of passenger cars that would consume the same amount of roadway capacity as a single large truck, bus, or recreational vehicle. These passenger car equivalents are denoted E_T for large trucks and buses and E_R for recreational vehicles, and are a function of roadway grades because steep grades will tend to magnify the poor performance of heavy vehicles as well as the sight distance problems caused by their larger dimensions (the visibility afforded to drivers in vehicles following heavy vehicles). For segments of freeway that contain a mix of grades, an extended segment analysis can be used as long as no single grade is steep enough or long enough to significantly impact the overall operations of the segment. As a guideline, an extended segment analysis can be used for freeway segments where no single grade that is less than 3% is more than 0.8 km long, or no single grade that is 3% or greater is longer than 0.4 km. If an extended segment analysis is used, the terrain must be generally classified according to the following definitions [Transportation Research Board 2010]:

> **Level terrain.** Any combination of horizontal and vertical alignment permitting heavy vehicles to maintain approximately the same speed as passenger cars. This generally includes short grades of no more than 2%.

> **Rolling terrain.** Any combination of horizontal and vertical alignment that causes heavy vehicles to reduce their speed substantially below those of passenger cars but does not cause heavy vehicles to operate at their limiting speed [$F_{net}(V) \neq 0$] for the given terrain for any significant length of time or at frequent intervals due to high grade resistance, as illustrated in Fig. 2.6.
>
> **Mountainous terrain.** Any combination of horizontal and vertical alignment that causes heavy vehicles to operate at their limiting speed for significant distances or at frequent intervals.

The passenger car equivalency factors for an extended segment analysis can be obtained from Table 6.5.

Table 6.5 Passenger Car Equivalents (PCEs) for Extended Freeway Segments

	Type of terrain		
Factor	Level	Rolling	Mountainous
E_T (trucks and buses)	1.5	2.5	4.5
E_R (RVs)	1.2	2.0	4.0

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Any grade that does not meet the conditions for an extended segment analysis must be analyzed as a separate segment because of its significant impact on traffic operations. In these cases, grade-specific PCE values must be used. Tables 6.6 and 6.7 provide these values for positive grades (upgrades). These tables assume typical large trucks (with average weight-to-power ratios between 75 and 90 kg/kW) and recreational vehicles (with average weight-to-power ratios between 20 and 40 kg/kW). Note that the equivalency factors presented in these tables increase with increasing grade and length of grade, but decrease with increasing heavy vehicle percentage. This decrease with increasing percentage is due to the fact that heavy vehicles tend to group together as their percentages increase on steep, extended grades, thus decreasing their adverse impact on the traffic stream.

Sometimes it is necessary to determine the cumulative effect on traffic operations of several significant grades in succession. For this situation, a distance-weighted average may be used if all grades are less than 4% or the total combined length of the grades is less than 1220 m. For example, a 2% upgrade for 305 m followed immediately by a 3% upgrade for 610 m would use the equivalency factor for a 2.67% upgrade [(2 × 305 + 3 × 610)/915] for 915 m or 0.915 km. For information on additional analysis situations involving composite grades, refer to the *Highway Capacity Manual* [Transportation Research Board 2010]. These situations include combining two or more successive grades when the grades exceed 4% or the combined length is greater than 1220 m, determining the length of a grade that starts or ends on a vertical curve, and determining the point of greatest traffic impact in a series of grades (for example, if a long 5% grade were immediately followed by a 2% grade, the end of the 5% grade would be used, as this would be the point of minimum vehicle speed).

Table 6.6 Passenger Car Equivalents (E_T) for Trucks and Buses on Specific Upgrades

Upgrade (%)	Length (km)	Percentage of trucks and buses								
		2	4	5	6	8	10	15	20	25
< 2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
≥ 2–3	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.8–1.2	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 1.2–1.6	2.0	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 1.6–2.4	2.5	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 2.4	3.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
> 3–4	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	2.0	2.0	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.8–1.2	2.5	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	> 1.2–1.6	3.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 1.6–2.4	3.5	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
	> 2.4	4.0	3.5	3.0	3.0	3.0	3.0	2.5	2.5	2.5
> 4–5	0.0–0.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	3.0	2.5	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.8–1.2	3.5	3.0	3.0	3.0	2.5	2.5	2.5	2.5	2.5
	> 1.2–1.6	4.0	3.5	3.5	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.6	5.0	4.0	4.0	4.0	3.5	3.5	3.0	3.0	3.0
> 5–6	0.0–0.4	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.5	4.0	3.0	2.5	2.5	2.0	2.0	2.0	2.0	2.0
	> 0.5–0.8	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	> 0.8–1.2	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0	3.0
	> 1.2–1.5	5.5	5.0	4.5	4.0	3.0	3.0	3.0	3.0	3.0
	> 1.6	6.0	5.0	5.0	4.5	3.5	3.5	3.5	3.5	3.5
> 6	0.0–0.4	4.0	3.0	2.5	2.5	2.5	2.5	2.0	2.0	2.0
	> 0.4–0.5	4.5	4.0	3.5	3.5	3.5	3.0	2.5	2.5	2.5
	> 0.5–0.8	5.0	4.5	4.0	4.0	3.5	3.0	2.5	2.5	2.5
	> 0.8–1.2	5.5	5.0	4.5	4.5	4.0	3.5	3.0	3.0	3.0
	> 1.2–1.6	6.0	5.5	5.0	5.0	4.5	4.0	3.5	3.5	3.5
	> 1.6	7.0	6.0	5.5	5.5	5.0	4.5	4.0	4.0	4.0

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Negative grades (downgrades) also have an impact on equivalency factors because the comparatively poor braking characteristics of heavy vehicles have a more deleterious effect on the traffic stream than the level-terrain case. Table 6.8 gives the passenger car equivalents for trucks and buses on downgrades. It is assumed that recreational vehicles are not significantly impacted by downgrades, and therefore downgrade values for E_R are drawn from the level-terrain column in Table 6.5.

Table 6.7 Passenger Car Equivalents (E_R) for RVs on Specific Upgrades

Upgrade (%)	Length (km)	Percentage of RVs								
		2	4	5	6	8	10	15	20	25
≤ 2	All	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
> 2–3	0.0–0.8	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.8	3.0	1.5	1.5	1.5	1.5	1.5	1.2	1.2	1.2
> 3–4	0.0–0.4	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
	> 0.4–0.8	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5	1.5
	> 0.8	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5	1.5
4–5	0.0–0.4	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	> 0.4–0.8	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 0.8	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
> 5	0.0–0.4	4.0	3.0	2.5	2.5	2.5	2.0	2.0	2.0	1.5
	> 0.4–0.8	6.0	4.0	4.0	3.5	3.0	3.0	2.5	2.5	2.0
	> 0.8	6.0	4.5	4.0	4.5	3.5	3.0	3.0	2.5	2.0

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Table 6.8 Passenger Car Equivalents (E_T) for Trucks and Buses on Specific Downgrades

Downgrade (%)	Length (km)	Percentage of trucks			
		5	10	15	20
< 4	All	1.5	1.5	1.5	1.5
4–5	≤ 6.4	1.5	1.5	1.5	1.5
4–5	> 6.4	2.0	2.0	2.0	1.5
> 5–6	≤ 6.4	1.5	1.5	1.5	1.5
> 5–6	> 6.4	5.5	4.0	4.0	3.0
> 6	≤ 6.4	1.5	1.5	1.5	1.5
> 6	> 6.4	7.5	6.0	5.5	4.5

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Once the appropriate equivalency factors have been obtained, the following equation is applied to arrive at the heavy-vehicle adjustment factor f_{HV}:

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)} \tag{6.5}$$

where

f_{HV} = heavy-vehicle adjustment factor,
P_T = proportion of trucks and buses in the traffic stream,
P_R = proportion of recreational vehicles in the traffic stream,
E_T = passenger car equivalent for trucks and buses, from Table 6.5, 6.6, or 6.8, and
E_R = passenger car equivalent for recreational vehicles, from Table 6.5 or 6.7.

As an example of how the heavy-vehicle adjustment factor is computed, consider a freeway with a 1.6-km 4% upgrade with a traffic stream having 8% trucks, 2% buses, and 2% recreational vehicles. Tables 6.6 and 6.7 must be used because the grade is too steep and long for Table 6.5 to apply. The corresponding equivalency factors for this roadway are $E_T = 2.5$ (for a combined truck and bus percentage of 10) and $E_R = 3.0$, as obtained from Tables 6.6 and 6.7, respectively. Also, from the given percentages of heavy vehicles in the traffic stream, $P_T = 0.1$ and $P_R = 0.02$. Substituting these values into Eq. 6.5 gives $f_{HV} = 0.84$, or a 16% reduction in effective roadway capacity relative to the base condition of no heavy vehicles in the traffic stream.

Driver Population Adjustment

Under base conditions, the traffic stream is assumed to consist of regular weekday drivers and commuters. Such drivers have a high familiarity with the roadway and generally maneuver and respond to the maneuvers of other drivers in a safe and predictable fashion. There are times, however, when the traffic stream has a driver population that is less familiar with the roadway in question (such as weekend drivers or recreational drivers). Such drivers can cause a significant reduction in roadway capacity relative to the base condition of having only familiar drivers.

To account for the composition of the driver population, the adjustment factor f_p is used, and its recommended range is 0.85–1.00. Normally, the analyst should select a value of 1.00 for primarily commuter (or familiar-driver) traffic streams. But for other driver populations (for example, a large percentage of tourists), the loss in roadway capacity can vary from 1% to 15%. The exact value of the driver population adjustment factor is dependent on local conditions such as roadway characteristics and the surrounding environment (possible driver distractions such as scenic views and the like). When the driver population consists of a significant percentage of unfamiliar users, judgment is necessary to determine the exact value of this factor. This usually involves collection of data on local conditions (for further information, see [Transportation Research Board 2010]).

6.4.5 Calculate Density and Determine LOS

With all the terms in the previous equations defined, these equations can now be applied to determine freeway level of service and freeway capacity. The final step before level of service can be determined is to calculate the density of the traffic stream. The alternative notation to Eq. 6.1 is shown in Eq. 6.6, which will be used in subsequent example problems (for consistency with the *Highway Capacity Manual*).:

$$D = \frac{v_p}{S} \qquad \textbf{(6.6)}$$

where

D = density in pc/km/ln,
v_p = flow rate in pc/h/ln, and
S = average passenger car speed in km/h.

The average passenger car speed is found by reading it from the *y*-axis of Fig. 6.2 for the corresponding flow rate (v_p) and free-flow speed. Once the density value is calculated, the level of service can be read from Table 6.1 or Fig. 6.2.

Application of the process for determining basic freeway segment capacity and level of service will now be demonstrated by example.

EXAMPLE 6.1 BASIC FREEWAY SEGMENT LOS WITH GENERAL TERRAIN CLASSIFICATION

A six-lane urban freeway (three lanes in each direction) is on rolling terrain with 3.3-m lanes, obstructions 0.6 m from the right edge of the traveled pavement, and nine ramps within 5 kilometers upstream and 5 kilometers downstream of the midpoint of the analysis segment. The traffic stream consists primarily of commuters. A directional weekday peak-hour volume of 2300 vehicles is observed, with 700 vehicles arriving in the most congested 15-min period. If the traffic stream has 15% large trucks and buses and no recreational vehicles, determine the level of service.

SOLUTION

Determine the free-flow speed according to Eq. 6.2.

$$FFS = 121.3 - f_{LW} - f_{LC} - 7.96TRD^{0.84}$$

with

$f_{LW} = 3.1$ km/h (Table 6.3),
$f_{LC} = 2.6$ km/h (Table 6.4), and
$TRD = \frac{9}{10} = 0.9$ ramps/km

$$FFS = 121.3 - 3.1 - 2.6 - 7.96(0.9)^{0.84} = 108.3 \text{ km/h}$$

Rounding this *FFS* value to the nearest *FFS* in Table 6.2 gives a *FFS* of 104.6 km/h. Determine the flow rate according to Eq. 6.3:

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

with

$PHF = \frac{2300}{700 \times 4} = 0.821$
$N = 3$ (given),
$f_p = 1.0$ (commuters), and
$E_T = 2.5$ (rolling terrain, Table 6.5).

From Eq. 6.5 we obtain:

$$f_{HV} = \frac{1}{1+0.15(2.5-1)} = 0.816$$

So,

$$v_p = \frac{2300}{0.821\times 3\times 0.816\times 1.0} = 1144.4 \rightarrow 1145 \text{ pc/h/ln}$$

Obtaining average passenger car speed from Fig. 6.2 for a flow rate of 1145 and a *FFS* of 104.6 km/h yields an *S* of 104.6 km/h. In this case, the average speed is still the same as the *FFS* because the flow rate is low enough such that it is still on the linear/flat part of the speed-flow curve.

Now, density can be calculated with Eq. 6.6:

$$D = \frac{1145}{104.6} = 10.9 \text{ pc/km/ln}$$

From Table 6.1, it can be seen that this corresponds to LOS B (6.8 [max density for LOS A] < 10.9 < 11.2 [max density for LOS B]). Thus, this freeway segment operates at level of service B.

This problem can also be solved graphically by applying Fig. 6.2. Using this figure, draw a vertical line up from 1145 pc/h/ln (on the figure's *x*-axis) and find that this line intersects the 104.6 km/h free-flow speed curve in the LOS B density region (the dashed diagonal lines).

EXAMPLE 6.2 **BASIC FREEWAY SEGMENT LOS WITH A SPECIFIC GRADE**

Consider the freeway and traffic conditions in Example 6.1. At some point further along the roadway there is a 6% upgrade that is 2.4 km long. All other characteristics are the same as in Example 6.1. What is the level of service of this portion of the roadway, and how many vehicles can be added before the roadway reaches capacity (assuming that the proportion of vehicle types and the peak-hour factor remain constant)?

SOLUTION

To determine the LOS of this segment of the freeway, we note that all adjustment factors are the same as those in Example 6.1 except f_{HV}, which must now be determined using an equivalency factor, E_T, drawn from the specific-upgrade tables (in this case Table 6.6). From Table 6.6, $E_T = 3.5$, which gives

$$f_{HV} = \frac{1}{1+0.15(3.5-1)} = 0.727$$

So,

$$v_p = \frac{2300}{0.821\times 3\times 0.727\times 1.0} = 1284.5 \rightarrow 1285 \text{ pc/h/ln}$$

From Fig. 6.2, the average passenger car speed (*S*) is still 104.6 km/h; thus

$$D = \frac{1285}{104.6} = 12.3 \text{ pc/km/ln}$$

which gives LOS C from Table 6.1 (11.2 [max density for LOS B] < 12.3 < 16.2 [max density for LOS C]).

To determine how many vehicles can be added before capacity is reached, the hourly volume at capacity must be computed. Recall that capacity corresponds to a volume-to-capacity ratio of 1.0 (the threshold between LOS E and LOS F). For a free-flow speed of 104.6 km/h, the capacity is 2350 pc/h/ln. Equation 6.3 is rearranged and used to solve for the hourly volume based upon this capacity:

$$v_p = \frac{V}{PHF\times N\times f_{HV}\times f_p} \Rightarrow 2350 = \frac{V}{0.821\times 3\times 0.727\times 1.0}$$

which gives $V = 4208$ veh/h. This means that about 1908 vehicles (4208 – 2300) can be added during the peak hour before capacity is reached. It should be noted that the assumption that the peak-hour factor will remain constant as the roadway approaches capacity is not very realistic. In practice it is observed that as a roadway approaches capacity, *PHF* gets closer to 1. This implies that the flow rate over the peak hour becomes more uniform. This uniformity is the result of, among other factors, motorists adjusting their departure and arrival times to avoid congested periods within the peak hour.

6.5 MULTILANE HIGHWAYS

Multilane highways are similar to freeways in most respects, except for a few key differences:

- Vehicles may enter or leave the roadway at at-grade intersections and driveways (multilane highways do not have full access control).
- Multilane highways may or may not be divided (by a barrier or median separating opposing directions of flow), whereas freeways are always divided.
- Traffic signals may be present.
- Design standards (such as design speeds) are sometimes lower than those for freeways.
- The visual setting and development along multilane highways are usually more distracting to drivers than in the freeway case.

Multilane highways usually have four or six lanes (both directions), have posted speed limits between 60 and 100 km/h, and can have physical medians, medians that are two-way left-turn lanes (TWLTLs), or opposing directional volumes that may not be divided by a median at all. Two examples of multilane highways are shown in Fig. 6.3.

The determination of level of service on multilane highways closely mirrors the procedure for freeways. The main differences lie in some of the adjustment factors and their values. The procedure we present is valid only for sections of highway that are not significantly influenced by large queue formations and dissipations resulting from traffic signals (this is generally taken as having traffic signals spaced 3.2 km apart or more), do not have significant on-street parking, do not have bus stops with high usage, and do not have significant pedestrian activity.

Table 6.9 provides the level-of-service criteria corresponding to traffic density, speed, volume-to-capacity ratio, and the maximum flow rates for multilane highways. A graphical representation of this table is provided in Fig. 6.4.

Figure 6.3 Examples of multilane highways.

Table 6.9 LOS Criteria for Multilane Highways

Criterion	LOS				
	A	B	C	D	E
			FFS = 96.5 km/h		
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	24.9
Average speed (km/h)	96.5	96.5	95.6	91.3	88.5
Maximum *v/c*	0.30	0.49	0.70	0.90	1.00
Maximum flow rate (pc/h/ln)	655	1080	1550	1980	2200
			FFS = 88.5 km/h		
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	25.5
Average speed (km/h)	55.0	55.0	54.9	52.9	51.2
Maximum *v/c*	0.29	0.47	0.68	0.88	1.00
Maximum flow rate (pc/h/ln)	600	990	1430	1850	2100
			FFS = 80.5 km/h		
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	26.7
Average speed (km/h)	50.0	50.0	50.0	48.9	47.5
Maximum *v/c*	0.27	0.45	0.65	0.86	1.00
Maximum flow rate (pc/h/ln)	545	900	1300	1710	2000
			FFS = 72.4 km/h		
Maximum density (pc/km/ln)	6.8	11.2	16.2	21.7	28.0
Average speed (km/h)	45.0	45.0	45.0	44.4	42.2
Maximum *v/c*	0.26	0.43	0.62	0.82	1.00
Maximum flow rate (pc/h/ln)	490	810	1170	1550	1900

Note: Density is the primary determinant of LOS. Maximum flow rate values are rounded to the nearest 5 passenger cars.

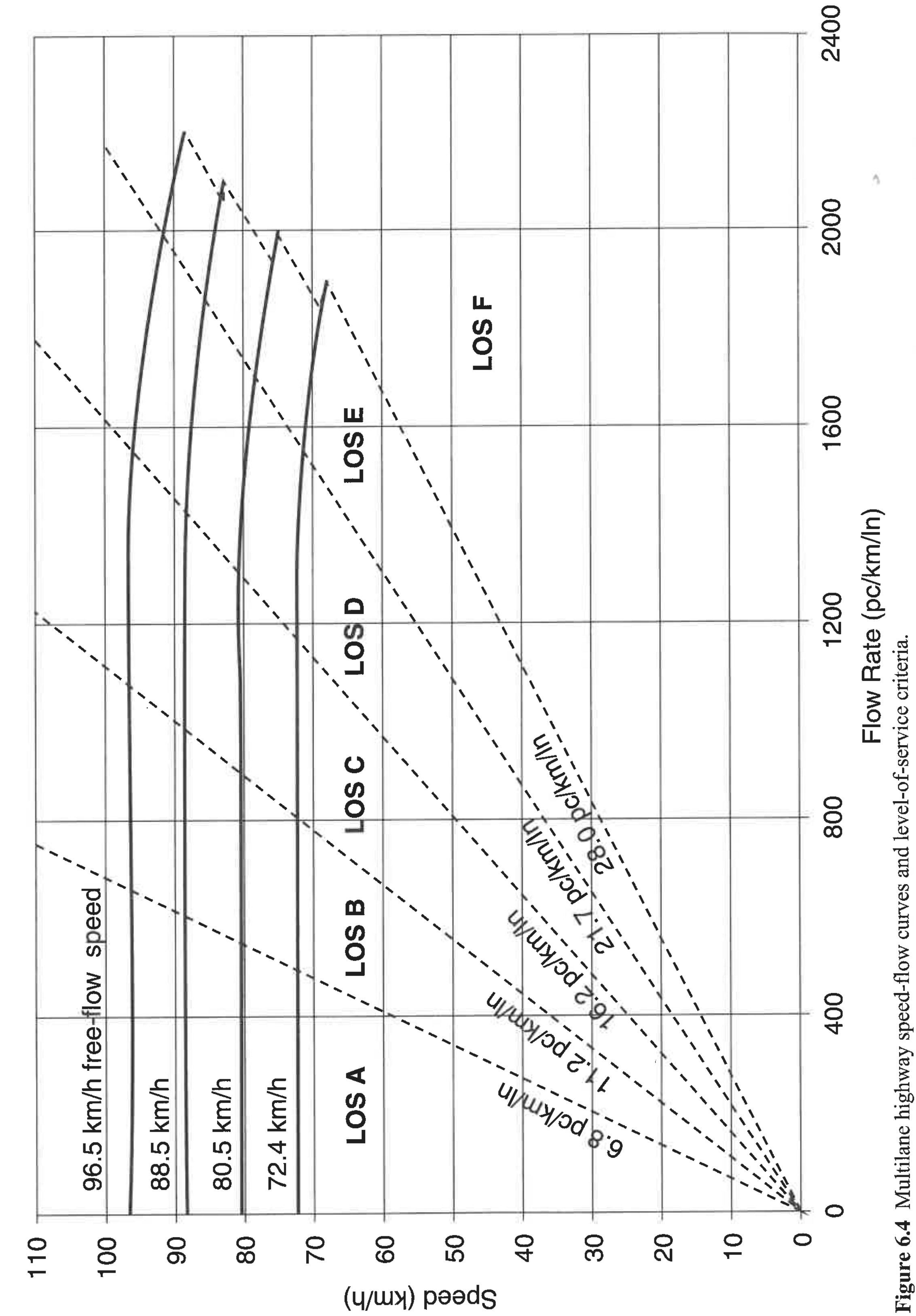

Figure 6.4 Multilane highway speed-flow curves and level-of-service criteria.
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6.5.1 Base Conditions and Capacity

The base conditions for multilane highways are defined as [Transportation Research Board 2010]

- 3.6-m minimum lane widths
- 3.6-m minimum total lateral clearance from roadside objects (right shoulder and median) in the travel direction
- Only passenger cars in the traffic stream
- No direct access points along the roadway
- Divided highway
- Level terrain (no grades greater than 2%)
- Driver population of mostly familiar roadway users
- Free-flow speed of 96.5 km/h or more

As was the case with the freeway level-of-service analysis, adjustments will have to be made when non-base conditions are encountered.

The capacity, *c*, for multilane highway segments, in pc/h/ln, is given in Table 6.10. From Table 6.9, note again that these capacity values correspond to the maximum service flow rate at LOS E and a *v*/*c* of 1.0.

6.5.2 Service Measure

Due to the large degree of similarity between multilane highway and freeway facilities, density is also the service measure (performance measure used for determining level of service) for multilane highways. However, the density threshold for LOS E varies by speed for multilane highways, as can be seen in Table 6.9. The density thresholds for levels of service A–D are the same for multilane highways and freeways.

6.5.3 Determine Free-Flow Speed

FFS for multilane highways is the mean speed of passenger cars operating in flow rates up to 1400 passenger cars per hour per lane (pc/h/ln). If *FFS* is to be estimated rather than measured, the following equation can be used, which takes into account the roadway characteristics of lane width, lateral clearance, presence (or lack) of a median, and access frequency:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A \qquad \textbf{(6.7)}$$

where

FFS = estimated free-flow speed in km/h,
$BFFS$ = estimated free-flow speed, in km/h, for base conditions,
f_{LW} = adjustment for lane width in km/h,
f_{LC} = adjustment for lateral clearance in km/h,
f_M = adjustment for median type in km/h, and
f_A = adjustment for the number of access points along the roadway in km/h.

As can be seen, this equation closely resembles Eq. 6.2 in the freeway section. Both include adjustments for lane width and lateral clearance, and the access frequency adjustment is similar to the ramp density adjustment. The main difference is that Eq. 6.7 also includes an adjustment for median type. The presence of a physical barrier or wide separation between opposing flows (such as a TWLTL) will lead to higher free-flow speeds than if there is no separation or physical barrier between opposing flows. This adjustment is not included for freeways since, by definition, all freeways are divided. As was the case for freeways, the HCM [Transportation Research Board 2010] recommends that the calculated free-flow speed be rounded to the nearest *FFS* values shown in Figure 6.4 and Table 6.9.

Table 6.10 Relationship Between Free-Flow Speed and Capacity on Multilane Highway Segments

Free-flow speed (km/h)	Capacity (pc/h/ln)
96.5	2200
88.5	2100
80.5	2000
72.4	1900

Source: Transportation Research Board, *Highway Capacity Manual* 2010. Washington, D.C. National Academy of Sciences. Table has been converted to SI units for use in this edition. Values from the source material are in U.S. Customary units.

As for *BFFS*, many factors can influence the free-flow speed, with the posted speed limit often being a significant one. For multilane highways, research has found that free-flow speeds, under base conditions, are about 11 km/h higher than the speed limit for 65- and 70-km/h posted-speed-limit roadways, and about 8 km/h higher for 80-km/h and higher posted-speed-limit roadways. The following sections describe the procedures for estimating the adjustment factor values.

Lane Width Adjustment

The same lane width adjustment factor values are used for multilane highways as are used for freeways. Thus, Table 6.3 should be used for multilane highways as well.

Lateral Clearance Adjustment

The adjustment factor for potentially restrictive lateral clearances (f_{LC}) is determined first by computing the total lateral clearance, which is defined as

$$TLC = LC_R + LC_L \qquad \textbf{(6.8)}$$

where

TLC = total lateral clearance in m,
LC_R = lateral clearance on the right side of the travel lanes to obstructions (retaining walls, utility poles, signs, trees, etc.), and
LC_L = lateral clearance on the left side of the travel lanes to obstructions.

For undivided highways, there is no adjustment for left-side lateral clearance because this is already taken into account in the f_M term (thus LC_L = 1.8 m in Eq. 6.8). If an individual lateral clearance (either left or right side) exceeds 1.8 m, 1.8 m is used in

Eq. 6.8. Finally, highways with TWLTLs are considered to have LC_L equal to 1.8 m. Once Eq. 6.8 is applied, the value for f_{LC} can be determined directly from Table 6.11.

Table 6.11 Adjustment for Lateral Clearance

*Total lateral clearance is the sum of the lateral clearances of the median (if greater than 1.8 m, use 1.8 m) and shoulder (if greater than 1.8 m, use 1.8 m). Therefore, for purposes of analysis, total lateral clearance cannot exceed 3.6 m.

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Total lateral clearance* (m)	Reduction in free-flow speed (km/h)	
	Four-lane highways	Six-lane highways
3.6	0.0	0.0
3.0	0.6	0.6
2.4	1.5	1.5
1.8	2.1	2.1
1.2	3.0	2.7
0.6	5.8	4.5
0.0	8.7	6.3

Median Adjustment

Values for the adjustment factor for median type, f_M, are provided in Table 6.12. This table shows that undivided highways have a free-flow speed that is 2.6 km/h lower than divided highways (which include those with two-way left-turn lanes).

Table 6.12 Adjustment for Median Type

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Median type	Reduction in free-flow speed (km/h)
Undivided highways	2.6
Divided highways (including TWLTLs)	0.0

Access Frequency Adjustment

The final adjustment factor in Eq. 6.7 is for the number of access points per kilometer, f_A. Access points are defined to include intersections and driveways (on the right side of the highway in the direction being considered) that significantly influence traffic flow, and thus do not generally include driveways to individual residences or service driveways at commercial sites. Adjustment values for access point frequency are provided in Table 6.13.

Table 6.13 Adjustment for Access-Point Frequency

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Access points/ kilometer	Reduction in free-flow speed (km/h)
0	0.0
6	4.0
12	8.0
18	12.0
≥ 24	16.0

6.5.4 Determine Analysis Flow Rate

The analysis flow rate for multilane highways is determined in the same manner as for freeways, using Eq. 6.3 and the remainder of the procedure outlined in Section 6.4.4. There is one minor difference for multilane highways—the guidelines for an extended segment analysis. An extended segment (general terrain type) analysis can be used for multilane highway segments if grades of 3% or less do not extend for more than 1.6 km or any grades greater than 3% do not extend for more than 0.8 km.

6.5.5 Calculate Density and Determine LOS

The procedure for calculating density and determining LOS for multilane highways is essentially the same as for freeways (see Section 6.4.5). Equation 6.6 is applied to arrive at a density. However, slightly different speed-flow curves and level-of-service criteria are used for multilane highways. Table 6.9 shows the level-of-service criteria for multilane highways, and Fig. 6.4 shows the corresponding speed-flow curves for multilane highways.

The average passenger car speed is found by reading it from the y-axis of Fig. 6.4 for the corresponding analysis flow rate (v_p) and free-flow speed. Once the density value is calculated, the level of service can be read from Table 6.9 or Fig. 6.4.

EXAMPLE 6.3 **MULTILANE HIGHWAY FREE-FLOW SPEED**

A four-lane undivided highway (two lanes in each direction) has 3.3-m lanes, with 1.2-m shoulders on the right side. There are four access points per kilometer, and the posted speed limit is 80 km/h. What is the estimated free-flow speed?

SOLUTION

This problem can be solved by direct application of Eq. 6.7 to arrive at an estimated free-flow speed:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

with

$BFFS$ = 88 km/h (assume FFS = posted speed + 8 km/h),
f_{LW} = 3.1 km/h (Table 6.3),
f_{LC} = 0.6 km/h (Table 6.11, with $TLC = 1.2 + 1.8 = 3.0$ from Eq. 6.8, with LC_L = 1.8 m because the highway is undivided),
f_M = 2.6 km/h (Table 6.12), and
f_A = 2.67 km/h (Table 6.13, by interpolation).

Substitution gives

$$FFS = 88 - 3.1 - 0.6 - 2.6 - 2.67 = \underline{\underline{79.03\text{ km/h}}}$$

which means that the more restrictive roadway characteristics relative to the base conditions result in a reduction in free-flow speed of 8.97 km/h. Note that for further analysis, this *FFS* value should be rounded to 80.5 km/h.

EXAMPLE 6.4 MULTILANE HIGHWAY LOS

A six-lane divided highway (three lanes in each direction) is on rolling terrain with 1.2 access points per kilometer and has 3.0-m lanes, with a 1.5-m shoulder on the right side and a 0.9-m shoulder on the left side. The peak-hour factor is 0.80, and the directional peak-hour volume is 3000 vehicles per hour. There are 6% large trucks, 2% buses, and 2% recreational vehicles. A significant percentage of nonfamiliar roadway users are in the traffic stream (the driver population adjustment factor is estimated as 0.95). No speed studies are available, but the posted speed limit is 90 km/h. Determine the level of service.

SOLUTION

We begin by determining *FFS* by applying Eq. 6.7:

$$FFS = BFFS - f_{LW} - f_{LC} - f_M - f_A$$

with

$BFFS$ = 98 km/h (assume FFS = posted speed + 8 km/h),
f_{LW} = 10.6 km/h (Table 6.3),
f_{LC} = 1.5 km/h (Table 6.11, with $TLC = 1.5 + 0.9 = 2.4$ from Eq. 6.8,
f_M = 0.0 km/h (Table 6.12), and
f_A = 0.8 km/h (Table 6.13, by interpolation).

Substitution gives

$$FFS = 98.0 - 10.6 - 1.5 - 0.0 - 0.8 = 85.1 \text{ km/h}$$

Rounding this *FFS* value to the nearest 8 km/h gives a *FFS* of 88.5 km/h. Next we determine the analysis flow rate using Eq. 6.3:

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

with

V = 3000 veh/h (given),
PHF = 0.80 (given),
N = 3 (given),
f_p = 0.95 (given),
E_T = 2.5 (Table 6.5), and
E_R = 2.0 (Table 6.5).

From Eq. 6.5, we find

$$f_{HV} = \frac{1}{1 + 0.08(2.5-1) + 0.02(2-1)} = 0.877$$

Substitution gives

$$v_p = \frac{3000}{0.8 \times 3 \times 0.877 \times 0.95} = 1500.3 \text{ pc/h/ln}$$

Using Fig. 6.4, for *FFS* = 88.5 km/h, note that the 1500.3-pc/h/ln flow rate intersects this curve in the LOS D density region. Therefore, this highway is operating at LOS D.

EXAMPLE 6.5 MULTILANE HIGHWAY CAPACITY

A local manufacturer wishes to open a factory near the segment of highway described in Example 6.4. How many large trucks can be added to the peak-hour directional volume before capacity is reached? (Add only trucks and assume that the *PHF* remains constant.)

SOLUTION

Note that *FFS* will remain unchanged at 88.5 km/h. Table 6.9 shows that capacity for *FFS* = 88.5 km/h is 2100 pc/h/ln. The current number of large trucks and buses in the peak-hour traffic stream is 240 (0.08 × 3000) and the current number of recreational vehicles is 60 (0.02 × 3000). Let us denote the number of new trucks added as V_{nt}, so the combination of Eqs. 6.3 and 6.5 gives

$$v_p = \frac{V + V_{nt}}{(PHF)(N)\left[\dfrac{1}{1 + \left(\dfrac{240 + V_{nt}}{V + V_{nt}}\right)(E_T - 1) + \left(\dfrac{60}{V + V_{nt}}\right)(E_R - 1)}\right](f_p)}$$

with

v_p = 2100 pc/h/ln,
V = 3000 veh/h (Example 6.4),
PHF = 0.80 (Example 6.4),
N = 3 (Example 6.4),
f_p = 0.95 (Example 6.4),
E_T = 2.5 (Example 6.4), and
E_R = 2.0 (Example 6.4).

$$2100 = \frac{3000 + V_{nt}}{(0.80)(3)\left[\dfrac{1}{1 + \left(\dfrac{240 + V_{nt}}{3000 + V_{nt}}\right)(2.5 - 1) + \left(\dfrac{60}{3000 + V_{nt}}\right)(2 - 1)}\right](0.95)}$$

which gives $\underline{\underline{V_{nt} = 547}}$, which is the number of trucks that can be added to the peak-hour volume before capacity is reached.