NCEES adopted the following code updates for the October 2018 PE Civil Transportation exam:


Please use these updates for the corresponding pages in your copy of *PE Civil Reference Manual, Sixteenth Edition* (CERM16).


PCI: PCI Design Handbook: Precast and Prestressed Concrete, Seventh ed., 2010. Precast/Prestressed Concrete Institute, Chicago, IL.

TRANSPORTATION DESIGN STANDARDS


The HCM uses average travel speed as the primary defining parameter. Except for LOS F, the average travel and average running speeds are identical.

### SPOT SPEED STUDIES

The analysis of spot speed data draws on traditional statistical methods. The **mean**, **mode**, and **median speeds** are commonly determined. Certain percentile rank speeds may also be needed. For example, the **50th percentile speed** (i.e., the median speed) and the **85th percentile speed** (i.e., the **design speed**) can be found from the frequency distribution. The **pace interval** (also known as the **pace range**, **group pace**, or just **pace**) is the 10 mph speed range containing the most observations.

### VOLUME PARAMETERS

There are several volume parameters, and not all parameters will be needed in every capacity investigation. It is important to note if volumes are for both directions combined, for all lanes of one direction, or just for one lane.

The **average daily traffic**, ADT, and **average annual daily traffic**, AADT, may be one- or two-way. The **design hour volume**, DHV, is evaluated for the design year. DHV is usually the 30th highest hourly expected volume in the design year, hence its nickname “30th hour volume.” It is not an average or a maximum. DHV is two-way unless noted otherwise. The ratio of DHV to AADT is designated as the **K-factor**. Values range from 0.07 to 0.15. (See HCM Exh. 3-9.)

\[
K = \frac{\text{DHV}}{\text{AADT}}
\]

The **directional factor**, D, is the percentage of the dominant peak flow direction. It can range from up to 80% for rural roadways at peak hours, to 50% for central business district traffic, though values between 55% and 65% are more common. Values range from 0.51 to 0.70, and a typical value of D is 0.60. (See HCM Exh. 3-10.) The **directional design hour volume**, DDHV, is calculated as the product of the directional factor, D (also known as the **directionality factor** and the **D-factor**), and DHV.

\[
\text{DDHV} = D(DHV) = DK(\text{AADT})
\]

The rate of flow, \(v\), is the equivalent hourly rate at which vehicles pass a given point during a given time interval of less than 1 hr, usually 15 min. The rate of flow changes every 15 min.

The **design capacity** is the maximum volume of traffic that the roadway can handle. The **ideal capacity**, \(c\), for freeways is considered to be 2400 passenger cars per hour per lane (pcphpl). Different values apply for other levels of service, speeds, and types of facilities. The actual flow rate, \(v\), can be used to calculate the **volume-capacity ratio**, \(v/c\), for a particular level of service, \(i\).

\[
\text{volume-capacity ratio}_i = \frac{v}{c}_i
\]

The maximum service flow rate is the capacity in passenger cars per hour per lane under ideal conditions for a particular level of service, \(i\).

\[
\text{flow rate} = c\left(\frac{v}{c}\right)_i
\]
12. VEHICLE EQUIVALENTS

It is convenient to designate a standard unit of measure for vehicular flow. Passenger cars make up the majority of highway traffic, so it is natural to use passenger cars as the unit. Other types of vehicles are converted to passenger car equivalents. Since trucks, buses, and recreational vehicles (known as RVs) take up more space on a roadway than cars and since they tend to travel more slowly up grades, they degrade the quality of travel more than the same number of cars would. Therefore, their traffic volumes are converted to equivalent passenger car volumes, \( E \), when computing flow.

Passenger car equivalents for trucks and RVs on grades depend on the percent grade, the length of the grade, the number of lanes, and the percentage of trucks and buses. Table 73.6 lists passenger car equivalents for general conditions. However, these values are applicable only to long sections of highways. For operation on specific grades of specific lengths, the \( HCM \) must be used.

High-occupancy vehicles (HOVs) include taxis, buses, and carpool vehicles. Special \( HCM \) procedures apply to HOV lanes.

Table 73.6 Passenger Car Equivalents on Extended General Freeway Segments

<table>
<thead>
<tr>
<th>terrain</th>
<th>( E_T ) (trucks and buses)</th>
<th>( E_R ) (RVs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>level</td>
<td>1.5</td>
<td>1.2</td>
</tr>
<tr>
<td>rolling</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>mountainous</td>
<td>4.5</td>
<td>4.0</td>
</tr>
</tbody>
</table>

*Primarily for use in determining approximate capacity during planning stages when specific alignments are not known.


13. FREeways

(Capacity analysis of basic freeway segments is covered in \( HCM \) Chap. 11.)

The process of determining level of service of a freeway section generally involves determining the vehicular density, \( D \). Freeway conditions are classified into levels of service A through F. Level A represents conditions where there are no physical restrictions on operating speeds. Since there are only a few vehicles on the freeway, operation at highest speeds is possible. However, the traffic volume is small. Level F represents stop-and-go, low-speed conditions with poor safety and maneuverability. The desired design condition is between levels A and F. Typically, levels B and C are chosen for initial design in rural areas, and levels C and D are used for initial design in suburban and urban areas.

\[
D = \frac{V_p}{S}
\]
The actual level of service is determined by comparing the actual density (in pcpmpl) with the density limits given in Table 73.7.

### Table 73.7 Levels of Service for Basic Freeway Sections

<table>
<thead>
<tr>
<th>Level</th>
<th>Density Range (pcpmpl)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0–11</td>
<td>Free flow with low volumes and high speeds</td>
</tr>
<tr>
<td>B</td>
<td>&gt; 11–18</td>
<td>Stable flow, but speeds are beginning to be restricted by traffic conditions</td>
</tr>
<tr>
<td>C</td>
<td>&gt; 18–26</td>
<td>Stable flow, but most drivers cannot select their own speed</td>
</tr>
<tr>
<td>D</td>
<td>&gt; 26–35</td>
<td>Approaching unstable flow, and maneuvering room is noticeably limited</td>
</tr>
<tr>
<td>E</td>
<td>&gt; 35–45</td>
<td>Unstable flow with short stoppages, and maneuvering room is severely limited</td>
</tr>
<tr>
<td>F</td>
<td>&gt; 45</td>
<td>Forced flow at crawl speeds; localized lines of vehicles</td>
</tr>
</tbody>
</table>

(Multiply pcpmpl by 0.621 to obtain pc/km/h.)


On freeways, the HCM suggests using estimated PHF values ranging from 0.85 to 0.90 for rural freeways, lower values for suburban freeways. Lower values are typical of rural and lower volume conditions, while higher values are typical of urban and suburban peak-hour conditions.

The *flow rate*, \( V \), can be calculated from the number of lanes in the analysis, \( N \); a factor to adjust for the presence of heavy vehicles such as buses, trucks, and recreational vehicles, \( f_{HV} \); and a factor to adjust for the effect of the driver population, \( f_p \). Figure 73.4 illustrates the speed-flow relationship between flow rate and the average passenger car. The passenger car equivalent flow rate for the peak 15 min is shown as \( v_p \) (pcpchl).

\[
V = v_p \cdot (PHF) \cdot f_{HV} \cdot f_p
\]

Table 73.8 shows maximum service flow rates for various levels of service.

### Table 73.8 Maximum Freeway Service Flow Rates for Various LOS

<table>
<thead>
<tr>
<th>FFS (mi/hr)</th>
<th>LOS A (pcpchl)</th>
<th>LOS B (pcpchl)</th>
<th>LOS C (pcpchl)</th>
<th>LOS D (pcpchl)</th>
<th>LOS E (pcpchl)</th>
</tr>
</thead>
<tbody>
<tr>
<td>75</td>
<td>820</td>
<td>1310</td>
<td>1750</td>
<td>2110</td>
<td>2400</td>
</tr>
<tr>
<td>70</td>
<td>770</td>
<td>1250</td>
<td>1690</td>
<td>2070</td>
<td>2400</td>
</tr>
<tr>
<td>65</td>
<td>710</td>
<td>1170</td>
<td>1630</td>
<td>2030</td>
<td>2350</td>
</tr>
<tr>
<td>60</td>
<td>660</td>
<td>1080</td>
<td>1560</td>
<td>2010</td>
<td>2300</td>
</tr>
<tr>
<td>55</td>
<td>600</td>
<td>990</td>
<td>1430</td>
<td>1900</td>
<td>2250</td>
</tr>
</tbody>
</table>

*All values are rounded to the nearest 10 pcpchl.


Selection of the *driver population adjustment factor*, \( f_p \), requires engineering judgment based on the characteristics of the drivers using the freeway. For example, freeways used by commuter traffic familiar with the route are assigned high values of \( f_p \): 1.0 for urban freeways and 0.975 for rural freeways. The value drops to 0.75–0.90 for weekend, recreational, and other types of traffic that are less familiar with the route, and thus, do not use the available space as efficiently.

A study specific to driver performance in freeway work zones has indicated that the driver population adjustment factor can be modeled as the product of two numerical factors, a *familiarity factor* (which is dependent on the familiarity and aggressiveness and adaptability of the driver) and a *behavior factor* (which is dependent on the aggressiveness and accommodation of
the driver). The results of the study are summarized in Table 73.9. While these results have not been incorporated into the \textit{HCM} or extended beyond capacity studies in freeway work zones, they do indicate a wide range of possible values and, in extreme cases, that $f_p$ can even exceed 1.0.

For analysis of a basic freeway segment, free-flow speed, FFS, is used. The free-flow speed is the mean speed of passenger cars measured during flows of less than 1000 pc/h/ln (pcphpl). Field measurement is the preferred way to measure FFS. However, when field measurements are not available (such as for future facilities or for proposed changes to existing facilities), FFS can be estimated using Eq. 73.17 (\textit{HCM} Eq. 11-1), with adjustments for lane width, $f_{LW}$; right-side lateral clearance, $f_{LC}$; and total ramp density, TRD, in ramps per mile. These factors are further described in \textit{HCM} Chap. 11.

\begin{equation}
\text{FFS} = 75.4 \text{ mi/hr} - f_{LW} - f_{LC} - (3.22 \text{ mi/hr}) \text{TRD}^{0.84}
\end{equation}

Due to the considerable variations between observed and predicted values, the \textit{HCM} recommends rounding FFS to the nearest 5 mph. Density thresholds for each LOS remain the same for all FFS criteria, so the maximum service flow varies for each LOS.

\begin{table}[h]
\centering
\begin{tabular}{|c|c|c|c|c|}
\hline
familiarity & adaptability & aggressiveness & accommodation & driver population factor \\
\hline
high & high & medium & high & 1.375 \\
\hline
medium & medium & medium & medium & 0.9 \\
\hline
low & low & low & low & 0.64 \\
\hline
\end{tabular}
\caption{Driver Population Adjustment Factors for Freeway Work Zone Capacity}
\end{table}

Example 73.3
A four-lane (two lanes in each direction) freeway passes through rolling terrain in an urban area. The freeway is constructed with 11 ft lanes and abutment walls 2 ft from the outer pavement edges of both slow lanes. The one-direction peak hourly volume during the weekday commute is 1800 vph. Traffic includes 3\% buses and 5\% trucks. The ramp density is one ramp per mile. The peak-hour factor is 0.90. The posted speed limit is 65 mph. (a) What is the passenger car equivalent flow rate per lane? (b) What is the speed during peak-hour travel? (c) What is the density? (d) What is the weekday peak-hour level of service?

\begin{align}
\text{Solution} \\
\text{(a) Trucks and buses have the same vehicle equivalents, so the } & \text{“truck” fraction is 8\%. There are no RVs. From Table 73.6, } E_T = 2.5. \text{ From Eq. 73.16,} \\
& f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + (0.08)(2.5 - 1)} = 0.892. \text{ From Eq. 73.15,} \\
& v_p = \frac{v}{(PHF)N_{HV}f_p} = \frac{1800 \text{ vph}}{(0.90)(2)(0.892)(1)} = 1120 \text{ pcphpl.} \\
& f_{LW} = 1.9 \text{ mi/hr} \text{ [HCM Exh. 73-8]} \\
& f_{LC} = 2.4 \text{ mi/hr} \text{ [HCM Exh. 73-9]} \\
& \text{TRD} = 1.0 \text{ ramp/mi} \text{ [HCM Exh. 73-10]} \\
& \text{FFS} = 75.4 \text{ mi/hr} - f_{LW} - f_{LC} - (3.22 \text{ mi/hr}) \text{TRD}^{0.84} \\
& = 75.4 \text{ mi/hr} - 1.9 \text{ mi/hr} - 2.4 \text{ mi/hr} - (3.22 \text{ mi/hr})(1.0)^{0.84} \\
& = 67.88 \text{ mi/hr} \text{ [say 70 mi/hr]} \\
\end{align}

\begin{align}
\text{(c) The density is} \\
& D = \frac{v_p}{S} = \frac{1120 \text{ pcphpl}}{65 \text{ pc/mi-ln (pcpmpl)}} \\
& = 17 \text{ pc/mi-ln (pcpmpl)} \\
\end{align}

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{Figure73.4.png}
\caption{Figure 73.4}
\end{figure}

\begin{align}
\text{(d) From Table 73.8, with a free-flow speed of 70 mph, the level of service (based on flow rate) is B.}
\end{align}

Example 73.4
An urban freeway segment is being designed for an AADT of 60,000 weekday commuters traveling at 60 mph in 12 ft lanes. Heavy trucks constitute 5\% of the total traffic. The directionality factor is 75\%. The \textit{K}-factor is 9.8\%. The freeway segment consists of 1 mi of
4% upward grade. The lateral clearances are 10 ft on the right and 6 ft on the left. The free-flow speed, FFS, is 65 mph. The peak hour factor, PHF, is 0.92. How many lanes are needed for LOS D?

Solution

Use Eq. 73.2 to convert AADT to design hourly volume.


\[
DDHV = DK(\text{AADT}) = (0.75)(0.098)\left(60,000 \text{ vph day}\right)
\]

\[
= 4410 \text{ vph}
\]

From HCM Exh. 11-11, for a 4% upgrade 1 mi in length with 5% trucks, the passenger car equivalent for trucks is 2.5. From Eq. 73.16,

\[
f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + (0.05)(2.5 - 1)}
\]

\[
= 0.990 \times 0.909
\]

From Table 73.8 with FFS = 65 mph and LOS D, the maximum service flow rate is 2030 pcphpl.

The adjustment factor for a driver population of weekday commuters is \( f_p = 1.0 \).

\[
V_{\text{2 lanes}} = v_p(\text{PHF})N_{fHV} = (2030 \text{ pc hr-ln})(0.92)(2 \text{ ln})(0.930)(1) = 3474 \text{ vph} \leq 4410 \text{ vph}
\]

\[
V_{\text{3 lanes}} = v_p(\text{PHF})N_{fHV} = (2030 \text{ pc hr-ln})(0.92)(3 \text{ ln})(0.930)(1) = 5211 \text{ vph} > 4410 \text{ vph}
\]

Three lanes are required.

14. MULTILANE HIGHWAYS

(Capacity analysis of multilane highways is covered in HCM Chap. 14.)

Multilane highways generally have four to six lanes and posted speed limits between 40 mph and 55 mph. The flow directions may be divided by a median, may be undivided with only a centerline separating them, or may have a two-way left-turn lane (TWLT). Multilane highways are typically located in suburban areas or between central cities along high-volume corridors. As such, they often are used by public transit, include bike lanes, and carry high traffic volumes. Access is generally through at-grade intersections.\(^4\) Signals are generally spaced close together in urban areas and several miles apart in suburban or rural areas. Segments with more frequently placed signals must be analyzed as interrupted flow urban streets.

While uninterrupted flow on multiline highways is similar to that on freeways, there are several different factors that must be accounted for. Various degrees of intersecting side traffic are present (such as from uncontrolled driveways and intersections, and from opposing flows on undivided cross sections). As a result, speeds and capacities on multiline highways are lower than on basic freeways with similar cross sections.

Multilane highways are not completely access controlled. The free-flow speed, FFS, is the theoretical speed of a vehicle under all actual conditions except interference by other vehicles. Free-flow speed is the theoretical speed when density is zero, but is essentially unchanged for densities up to 1400 pcphpl. Free-flow speed is determined from the base free-flow speed, BFFS, with adjustments for median type, \( f_M \); lane width, \( f_w \); total lateral clearance, \( f_{LC} \); and density of access points, \( f_A \). The base free-flow speed is covered in HCM Chap. 14, but is approximately 5 mph greater than the speed limit for 50 mph and 55 mph speed limits. For speed limits less than 50 mph, it is approximately 7 mph greater. Values of the adjustments are obtained from Table 73.10 and the appropriate tables in HCM Chap. 14. (Though similar in concept to freeway adjustments, some highway values are different. Accordingly, different symbols are used.)

\[
\text{FFS} = \text{BFFS} - f_M - f_LW - f_{LC} - f_A 73.18
\]

The 15 min passenger car equivalent flow rate, \( v_p \), in pcphpl is given by Eq. 73.19. \( V \) is the volume of vehicles passing a point each hour, \( f_{HV} \) is the same heavy-vehicle factor used in freeway analysis, and PHF is the peak hour factor. Where specific local data are not available, the HCM recommends reasonable estimates of PHF of 0.75 to 0.95. Lower values are typical of low volume rural conditions, and higher values are typical of urban and suburban peak-hour conditions. For congested conditions, a PHF of 0.95 should be used. The driver population adjustment factor, \( f_p \), ranges from 0.85 to 1.00. The HCM recommends using a value of 1.00, which represents familiar (e.g., commuter) drivers, unless there is sufficient evidence to use a lower value.

\[
v_p = D_m(\text{FFS}) = \frac{V}{N(\text{PHF})f_{HV}f_p} 73.19
\]

Example 73.5

An undivided suburban multilane highway segment with four 11 ft lanes (two in each direction) is used by weekday commuters. There is no median, and the two lanes in each direction are separated by a striped

---

\(^4\)Segments between signals that are placed 2 mi or more apart function similarly to freeways and can be considered as such for analysis purposes.
The estimated free-flow speed under ideal conditions is 60 mph. The segment contains 4% upgrades and downgrades 0.8 mi long. The fraction of recreational vehicles on this segment is essentially zero. However, the fraction of trucks is 10%. The lateral clearance on the right-hand side of the slow lane is 6 ft from the pavement edge. There is a 2 ft shoulder on each outside lane. There are no points of entry to the highway segment.

(a) What is the hourly volume in the upgradedirection for LOS C? (b) What is the maximum capacity of the downgrade section? (c) What is the LOS in the downgrade section during the morning peak if the peak-hour traffic volume is 1300 vph?

Solution

(a) The base free-flow speed is given as BFFS = 60 mph.

The median type adjustment factor for undivided highways is found from HCM Exh. 14-10, as \( f_M = 1.6 \).

The lane width adjustment factor for 11 ft lanes is found from HCM Exh. 14-8, as \( f_LW = 1.9 \).

For undivided highways with only a striped centerline, the left side clearance is zero. However, the median factor accounts for the proximity of the two opposing lanes. The left-edge lateral clearance is taken as 6 ft per the HCM. The total lateral clearance is 6 ft + 2 ft = 8 ft. The adjustment factor for lateral clearance for four-lane highways is found in HCM Exh. 14-9, as \( f_C = 0.9 \).

If there are no access points, HCM Exh. 14-11 gives the access-point density adjustment factor as \( f_A = 0 \).

(b) Use Eq. 73.18.

\[
FFS = BFFS - f_M - f_LW - f_C - f_A
\]

\[
= 60 \text{ mi/hr} - 1.6 \text{ mi/hr} - 1.9 \text{ mi/hr} - 0.9 \text{ mi/hr}
\]

\[
= 55.6 \text{ mi/hr (mph)}
\]

Round to 55 mph in order to use data from Table 73.10. The maximum density at LOS C is 26 pcpmpl.

Use Eq. 73.19.

\[
v_p = D_m(FFS) = \left( \frac{26 \text{ pc}}{\text{mi-lane}} \right) \left( \frac{55.6 \text{ mi}}{\text{hr}} \right)
\]

\[
= 1446 \text{ pc/hr-lane (pcphpl)}
\]

From Exh. 14-13 of the HCM, the passenger car equivalent of trucks on \( 4\% \) grade 0.8 mi long with 10% trucks is \( E_T = 2.5 \). From Eq. 73.16, the heavy-vehicle adjustment factor is

\[
f_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + (0.10)(2.5 - 1)}
\]

\[
= 0.833
\]

Assume a value of 0.95 for the peak hour factor as recommended by the HCM. Solve Eq. 73.19 for the volume.

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

\[
= \left( 1446 \frac{\text{ pc}}{\text{hr-lane}} \right)(2 \text{ lanes})(0.95)(0.833)(1.0)
\]

\[
= 2890 \text{ pc/hr (vph)}
\]
Highways where slower travel is expected by drivers (i.e., access is favored over mobility) are referred to as Class II highways. Examples of Class II highways include sightseeing routes, scenic byways, routes through rugged terrain, and highways that connect to Class I highways. Class II highways are generally used on relatively short trips, the beginning and end of longer trips, or trips for which sightseeing plays a significant role.

Class III two-lane highways are segments of Class I or Class II two-lane highways that pass through small towns or developed recreational areas. They serve moderately developed areas where local traffic mixes with through traffic, and where unsignalized access points occur more frequently. Class III two-lane highways may include longer segments passing through relatively spread-out recreational areas and areas with increased roadside densities.

Under base conditions, the capacity of a two-lane highway is 1700 pcp/h for each direction of travel. This capacity is essentially independent of the directional distribution of traffic on the highway, although for extended lengths of two-lane highway, the capacity will not exceed 3200 pcp for both directions of travel combined. For short lengths of two-lane highway—such as tunnels and bridges—a combined capacity of 3200 pcp to 3400 pcp for both directions of travel may be achieved.

The performance criteria (i.e., measures of effectiveness used to determine LOS) for highways include:

- **average travel speed** (ATS): the average time required to travel a highway segment divided by the segment length, an indicator of mobility.
- **percent time spent following** (PTSF): the average percentage of time that vehicles must travel in platoons behind slower vehicles due to the inability to pass, reflecting the convenience of travel. PTSF is sometimes calculated as the percentage of vehicles traveling at headways of less than 3.0 sec, or as the percentage of vehicles traveling in platoons.
- **percent of free-flow speed** (PFFS): the ability vehicles have to travel at, or near, the posted speed limit

The performance criteria of Class I, two-lane highways in non-mountainous terrain with no traffic signals include ATS and PTSF. Performance of Class II highways is determined only by PTSF. Performance of Class III highways is determined only by PFFS as these highways are generally limited in length and have lower posted speed limits. Table 73.11 lists the LOS criteria for two-lane highways.

If the actual volume of traffic, \( v_p \), for both directions is greater than 3200 pcp/h, then the roadway is over capacity (i.e., saturated), and LOS is F. If \( v_p \) in a single direction is greater than 1700 pcp/h, LOS is also F. In these three cases, no further analysis is needed to determine how the highway is performing.
LOS for two-lane highways can be evaluated for general segments (e.g., level or rolling terrain), two-way segments with specific upgrades and downgrades, or one-way directional segments. Specific upgrades and downgrades are those that have grades of 3% or greater and lengths of 0.6 mi or greater. The one-way directional segment methodology treats heavy vehicles differently and can only be used in level or rolling terrain, not in conditions that qualify for specific upgrades and downgrades. Directional segment methodology is not covered in this chapter.

In addition to having different methodologies for general highway segments and specific upgrades and downgrades, the various adjustment factors for ATS and PTSF are different. Table 73.12 lists the HCM exhibits for each case.

Table 73.12 HCM Exhibits for Two-Lane Highway Adjustments

<table>
<thead>
<tr>
<th>general sections</th>
<th>ATS</th>
<th>PTSF</th>
</tr>
</thead>
<tbody>
<tr>
<td>grade adjustment, $f_g$</td>
<td>15-9</td>
<td>15-16</td>
</tr>
<tr>
<td>truck equivalent, $E_T$</td>
<td>15-11</td>
<td>15-18</td>
</tr>
<tr>
<td>recreational vehicle, $E_R$</td>
<td>15-11</td>
<td>15-18</td>
</tr>
</tbody>
</table>

specific upgrades

| grade adjustment, $f_g$ | 15-10 | 15-17 |
| truck equivalent, $E_T$ | 15-12 | 15-19 |
| recreational vehicle, $E_R$ | 15-13 | 15-19 |

The free-flow speed, FFS, is the speed of traffic under low-low conditions. If ATS methodology will be used, FFS can be determined two ways: The average speed, $S_{FM}$, based on field measurements (observations) can be adjusted for flow rate and heavy vehicles, or FFS can be calculated from the base free-flow speed, BFFS. The base free-flow speed is generally specified by the agency, and usually will be somewhat higher (e.g., 10 mi/hr faster) than the posted speed limit or the design speed. In Eq. 73.20, $v_l$ is the observed flow rate for the period corresponding to $S_{FM}$. In Eq. 73.21, $f_{LS}$ is the lane and shoulder adjustment factor from HCM Exh. 15-17, and $f_a$ is the adjustment for access point density (intersections and driveways) from HCM Exh. 15-18.

$$FFS = S_{FM} + 0.00766 \frac{v_l}{f_{HV,ATS}}$$  \hspace{1cm} 73.20

$$FFS = BFFS - f_{LS} - f_a \quad \text{[estimated BFFS]} \quad 73.21$$

The demand flow rate, $v_l$ (passenger car flow rate for speed), is the adjusted hourly demand in passenger car equivalents for the peak 15 minutes, calculated from the hourly volume, $V_i$. The subscript $i$ is used to denote either the analysis direction, $d$, or the opposing direction, $a$ (e.g., $v_d$ or $v_o$). For clarity, $v_{ATS}$, $f_{ATS}$, and $f_{HV,ATS}$ for ATS use are distinguished from their percent time spent following counterparts $v_{PTSF}$, $f_{PTSF}$, and $f_{HV,PTSF}$ by the use of subscripts ATS and PTSF, but have the same meanings otherwise. Equation 73.22 is only used for Class I highways.

$$v_{ATS} = \frac{v_i}{(PHF)f_{ATS}f_{HV,ATS}} \quad [ATS] \quad 73.22$$

Similarly, the demand flow rate for percent time spent following is

$$v_{PTSF} = \frac{v_i}{(PHF)f_{PTSF}f_{HV,PTSF}} \quad [PTSF] \quad 73.23$$

The PHF is used to convert hourly volumes to flow rates and represents the hourly variation in traffic flow. If the demand volume is measured in 15 min increments, it is unnecessary to use the PHF to convert to flow rates. Therefore, since two-lane highway analysis is based on demand flow rates for a peak 15 min period within the analysis hour (usually the peak hour), the PHF in Eq. 73.22 and Eq. 73.23 is given a value of 1.00.

The average travel speed in the analysis direction, $ATS_d$, is estimated from the FFS, the demand flow rate, the opposing flow rate, and the adjustment factor for the percentage of no-passing zones in the analysis direction, $f_{np}$, as given in HCM Exh. 15-15. Equation 73.24 only applies to Class I and Class III two-lane highways.

$$ATS_d = FFS - 0.00766(v_{ATS} + v_{PTSF}) - f_{np}$$  \hspace{1cm} 73.24

If the PTSF methodology is used, the formula for the demand flow rate, $v_{ATS}$, is the same, although different values of $f_g$ and $f_{HV}$ apply. The base percent time spent following in the analysis direction, BPTSF, is given by Eq. 73.25, where $a$ and $b$ are constants as found from Table 73.13 (HCM Exh. 15-20).

$$BPTSF_d = (1 - e^{-ab}) \times 100 \% \quad 73.25$$
Table 73.13 Coefficients for Estimating BPTSF

<table>
<thead>
<tr>
<th>opposing demand flow rate, ( v ) (pc/h)</th>
<th>coefficient ( a )</th>
<th>coefficient ( b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 200 )</td>
<td>-0.0014</td>
<td>0.973</td>
</tr>
<tr>
<td>400</td>
<td>-0.0022</td>
<td>0.923</td>
</tr>
<tr>
<td>600</td>
<td>-0.0033</td>
<td>0.870</td>
</tr>
<tr>
<td>800</td>
<td>-0.0045</td>
<td>0.833</td>
</tr>
<tr>
<td>1000</td>
<td>-0.0049</td>
<td>0.829</td>
</tr>
<tr>
<td>1200</td>
<td>-0.0054</td>
<td>0.825</td>
</tr>
<tr>
<td>1400</td>
<td>-0.0058</td>
<td>0.821</td>
</tr>
<tr>
<td>( \geq 1600 )</td>
<td>-0.0062</td>
<td>0.817</td>
</tr>
</tbody>
</table>

Note: Straight-line interpolation of \( a \) to the nearest 0.0001 and \( b \) to the nearest 0.001 is recommended.


The percent time spent following, PTSF, is calculated from the BPTSF, the adjustment factor for the percentage of no-passing zones in the analysis direction, \( f_{np} \) (HCM Exh. 15-21), and the demand flow rates in the analysis and opposing directions.

\[
PTSF_d = BPTSF_d + f_{np,PTSF} \left( \frac{v_d,PTSF}{v_d,PTSF + v_o,PTSF} \right)
\]

Equation 73.26 is used to determine LOS for Class I and Class II highways only. Class III highways use the percent free-flow speed, PFFS, instead.

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

16. URBAN STREETS

Urban streets are roadways that have fixed traffic interruptions at frequent intervals. These interruptions include signaled intersections spaced less than 2 mi (3.2 km) apart, roundabouts, and transit facilities. HCM Chap. 16 and Chap. 17 provide integrated methodologies for quantitatively evaluating the performance of urban streets, including the LOS for each transportation mode (i.e., automobile, pedestrian, bicycle, and mass transit).

17. SIGNALIZED INTERSECTIONS

(Capacity analysis of signalized intersections is covered in HCM Chap. 18.)

Signalized intersections are controlled by signals operating in two or more phases. Each phase consists of three intervals: green, amber (i.e., “yellow”), and red. For a typical intersection with two streets crossing at 90\(^\circ\) to each other, a two-phase signal is one that has one phase for each axis of travel (e.g., one phase of north-south movements and one phase of east-west movements). A three-phase signal provides one of the roads with a left-turn phase. In a four-phase signal, both roads have left-turn phases. Figure 73.5 shows typical elements of an intersection.

Level of service for signalized intersections is defined in terms of control delay in the intersection. Control delay consists of only the portion of delay attributable to the control facility (e.g., initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay), but not geometric delay or incident delay.

Geometric delay is caused by the need for vehicles to slow down in order to negotiate an intersection. For example, a vehicle would need to slow down in order to enter a separate left-turn lane or negotiate a roundabout, regardless of the traffic already in the intersection. Queuing delay is the delay caused by the need to slow down in order to avoid other vehicles already present. All or parts of queuing delay may be included in geometric delay. For example, the time a driver takes to determine if there is traffic that needs to be avoided overlaps both geometric and queuing delay. Incident delay occurs when there is an incident (i.e., an accident). There are nine general categories of incidents (fatal injury, personal injury, property damage, mechanical/electrical failure, other stalls, flat tires, abandoned vehicles, debris on roadway, and everything else). Incident delay includes all times for which lanes and shoulders are blocked, including response and clearance times.

Delay can be measured in the field, or it can be estimated using procedures as outlined in the HCM. LOS criteria are stated in terms of control delay per vehicle...
The capacity of an approach or lane group is given by Eq. 73.28. \( s \) is the saturation flow rate in vphg for lane group \( i \) (i.e., in vphgpl), which is the flow rate per lane at which vehicles can pass through a signalized intersection in a stable moving queue. \( N \) is the number of lanes. \( g_i/C \) is the effective green ratio for the lane group.

\[
\frac{c_i}{N} = N_s \left( \frac{g_i}{C} \right)
\]  

Eq. 73.28

The ratio of flow to capacity \((v/c)\) is known as the degree of saturation and volume-capacity ratio. For convenience and consistency with the literature, the degree of saturation for lane group or approach \( i \) is designated as \( X_i \) instead of \((v/c)_i \), which might be confused with level of service \( i \). When the flow rate equals the capacity, \( X_i \) equals 1.00; when flow rate is zero, \( X_i \) is zero.

\[
X_i = \left( \frac{v}{c} \right) = \frac{v_i}{N_s g_i/C} = \frac{v_i C}{N_s g_i}
\]  

Eq. 73.30

When the overall intersection is to be evaluated with respect to its geometry and the total cycle time, the concept of the critical \( v/c \) ratio, \( X_c \), is used. The critical \( v/c \) ratio is usually obtained for the overall intersection considering only the critical lane groups or approaches. For a typical cross intersection with a two-, three-, or four-phase signal, once the total cycle length is selected, the time for each phase is proportioned according to the critical ratios, \( X_c \), of each phase. Other factors can affect the detail adjustment of the phase length such as lost time, pedestrian crossings, and approach conditions. In Eq. 73.31, \( C \) is the total cycle length and \( L \) is the lost time per cycle. Time is lost when the intersection is not used effectively as during start-up through the intersection. Lost time includes start-up time and some of the amber signal time. In conservative studies, lost time includes all of the amber signal time.

\[
X_c = \left( \frac{C}{C-L} \right) \sum_{i \in ci} y_{ci}
\]  

Eq. 73.31

The cycle lost time, \( L \), is found from Eq. 73.32, where \( l \) is the phase lost time.

\[
L = \sum_{i \in ci} l_{ci}
\]  

Eq. 73.32

Equation 73.31 can be used to estimate the signal timing for the intersection if a critical \( v/c \) ratio is specified for the intersection. This equation can also be used to estimate the overall sufficiency of the intersection by substituting the specified maximum permitted cycle length and determining the resultant critical \( v/c \) ratio for the intersection.
When the critical $v/c$ ratio is less than 1.00, the cycle length provided is adequate for all critical movements to pass through the intersection if the green time is proportionately distributed to the different phases. If the total green time is not proportionately distributed to the different phases, it is possible to have a critical $v/c$ ratio of less than 1.00, but one or more individual oversaturated movements may occur within a cycle.

**Saturation flow rate, $s$,** is defined as the total maximum flow rate on the approach or group of lanes that can pass through the intersection under prevailing traffic and roadway conditions when 100% of the effective green time is available. Saturation flow rate is expressed in units of vehicles per hour of effective green time (vphg) for a given lane group.

Saturation flow rate is calculated from the ideal saturation flow rate, $s_i$, which is assumed to be 1900 passenger cars per hour of green per lane (abbreviated “pcphgl”). Adjustments are made for lane width, $f_w$, heavy vehicles, $f_{HV}$; grade, $f_g$; existence of parking lanes, $f_p$; stopped bus blocking, $f_{bb}$; type of area, $f_a$; lane utilization, $f_U$; right-hand turns, $f_{RT}$; left-hand turns, $f_{LT}$; pedestrians and bicycles turning left and right, $f_{Lpb}$ and $f_{Rpb}$. All of the adjustments are tabulated in the HCM. **REPLACE FORMULA**

$$ s = s_i N f_w f_{HV} f_p f_{bb} f_A f_U f_{RT} f_{LT} f_{Lpb} f_{Rpb} $$

**Example 73.6**

A signalized intersection, without pedestrian access and located in a central business district, has an approach with two 11 ft lanes on a 2% downgrade. 10% of the traffic consists of heavy trucks, but there are no buses or RVs. Both lanes are through lanes; no turns are permitted.

There are no parking lanes. Arrivals to the intersection are random. What is the saturation flow rate of the approach?

**Solution**

Refer to the *HCM* to obtain the required adjustment factors.

$$ s_o = 1900 \text{ pcphgl} \quad [HCM \ p. 16-26] $$

$$ N = 2 \text{ lanes} \quad 19-20 $$

$$ f_w = 1.00 \quad [HCM \ Exh. 18-13] $$

$$ f_{HV} = 2.0 \quad [HCM \ p. 19-36] \quad 19-19 $$

From HCM Eq. 18-6,

$$ f_{HV} = \frac{100 - 0.79 P_{HV} - 2.07 P_g}{100} $$

From HCM Eq. 18-7,

$$ f_p = 1.00 \text{ for no parking} \quad [HCM \ p. 18-37] $$

$$ f_{bb} = 1.00 \text{ for no bus blocking} \quad [HCM \ p. 18-37] $$

$$ f_a = 0.900 \text{ for a central business (CBD)} \quad [HCM \ p. 18-47] $$

$$ f_{LU} = 1.0 \quad [HCM \ p. 18-38] $$

There is no pedestrian access and no turns are permitted, so

$$ f_{RT} = 1.000 $$

$$ f_{LT} = 1.000 $$

$$ f_{Lpb} = 1.000 $$

$$ f_{Rpb} = 1.000 $$

**REPLACE FORMULA**

From Eq. 73.33,

$$ s = s_o N f_w f_{HV} f_p f_{bb} f_A f_U f_{RT} f_{LT} f_{Lpb} f_{Rpb} $$

$$ = \left( 1900 \frac{pc}{\text{hrg-lane}} \right) \left( 2 \text{ lanes} \right) \left( 1.00 \right) \left( 0.900 \right) \left( 1.010 \right) \times \left( 1.000 \right) \left( 1.000 \right) $$

$$ = 3140 \text{ pcphgl} $$

$$ 3323 $$
an average day for any consecutive four hours per day or for any four consecutive 15-minute periods falls above its respective curve.

- **Warrant 5, School Crossing:** A large number of student pedestrians crossing a major street may justify signalization. The warrant requires a minimum of 20 students during the highest crossing hour and the determination that there are fewer adequate crossing gaps in the traffic stream than there are minutes in the period when children are crossing. There cannot be another traffic control signal closer than 300 ft (90 m). All other methodologies for providing crossing opportunities should be considered before applying Warrant 5.

- **Warrant 6, Coordinated Signal System:** Signals can be justified if they will induce desirable platooning and progressive movement of vehicles. This warrant can be applied to streets on which the traffic is predominantly one-way or two-way, as long as resulting signals are at least 1000 ft (300 m) apart, and the existing signals are so far apart that vehicular platooning is not achieved.

- **Warrant 7, Crash Experience:** This warrant is based on crash frequency. Three conditions are specified, all of which must be met: (A) Alternative remediation and enforcement efforts have been unsuccessful in reducing crash frequency. (B) Five or more crashes involving reportable personal injury or property damage and that would have been prevented by signalization have occurred within a 12-month period. (C) The intersection experiences a minimum level of traffic during each of any eight hours of an average day. (See the MUTCD.)

- **Warrant 8, Roadway Network:** Signalization may be considered to encourage concentration and organization of traffic flow on a roadway network. Routes that can be considered for this warrant must be part of a current or future highway system that serves as the principal roadway network for through-traffic flow or includes highways entering or serving a city. This warrant requires either an intersection volume of 1000 vehicles per hour during the peak hour of a typical weekday and is expected to meet Warrants 1, 2, and 3 during an average weekday in the future, or the intersection has or soon will have a volume of at least 1000 vehicles per hour during each of any five hours of the weekend (i.e., Saturday or Sunday).

### 21. FIXED-TIME CYCLES

Fixed-time controllers are the least expensive and simplest to use. They are most efficient only where traffic can be accurately predicted. Fixed-time controllers are necessary if sequential intersections or intersections spaced less than 1/2 mi (0.8 km) apart are to be coordinated.

In general, the fixed signal cycle length should be between 35 sec and 120 sec. Green cycle lengths with fixed-time controllers should be chosen to clear all waiting traffic in 95% of the cycles. Usually, the 85th percentile speed is used in preliminary studies. Since the green cycle must handle peak loads, level of service is sacrificed during the rest of the day, unless the cycle length is changed during the day with multi-dial controllers.

Amber time is usually 3–6 sec. Six seconds may be used for higher speed roadways. A short all-red clearance interval may be provided after the amber signal to clear the intersection. It is also necessary to check that pedestrians can cross the intersection in the available walk time. (See Sec. 73.25.) A short all-red clearance interval is suggested after the green walk signal terminates.

Determining cycle lengths of signalized intersections is covered in HCM Chap. 18. Queuing models and simulation can also be used to determine or check cycle lengths in complex situations.

### 22. TIME-SPACE DIAGRAMS

To minimize the frustration of drivers who might otherwise have to stop at every traffic signal encountered while traveling in a corridor, it is desirable to coordinate adjacent fixed-time signals. With alternate mode operation, every other signal will be green at the same time. By the time a vehicle moving at a specific speed has traveled the distance between two adjoining signals, the signal being approached will turn green. For double-alternate mode operation, two adjacent pairs of signals will have the same color (i.e., red or green), while the following two adjacent signals will have the opposite color. Alternate mode is preferred. Unless all of the signals are equidistant, it is unlikely that all inconvenience will be eliminated from the traffic stream.

Signal coordination is essentially achieved by setting the controller’s offset. Offset is the time from the end of one controller’s green cycle to the end of the next controller’s green cycle. The following graphical procedure can be used to establish the offset by drawing time-space diagrams (space-time diagrams). The horizontal axis represents distance (typical scales are 1 in:100 ft or 1 in:200 ft), and the vertical axis represents time (usually in seconds). (See Fig. 73.6.)

**step 1:** Draw the main and intersecting streets to scale along the horizontal scale. When signals are separated by short distances, the cross-street widths should be drawn accurately.

**step 2:** Assume or obtain the actual average travel speed along the main street. (This is rarely the posted speed limit.) Starting at the lower left corner, draw a diagonal line representing the average speed.
The next car will require 3 sec more. Subsequent cars between the detector and stop line require 2 1/2 sec. Studies have shown the average start-up lot time and average arrival headway to be approximately half of these values. However, these values accommodate the slowest of drivers.

- **Vehicle period:** The vehicle period must be long enough to allow a car crossing the detector (moving at the slowest reasonable speed) to get to the intersection before the amber signal appears. It is not necessary to have the vehicle get entirely through the intersection during the green, since the vehicle period will provide additional time. In a 30 mph (50 km/h) zone, a speed of approximately 20 mph (30 km/h) into the intersection is reasonable.

- **Maximum period:** The maximum period is the maximum of 3–6 sec for the amber period and 9–10 sec for the green period, depending on the vehicle and pedestrian flows. The amber period is determined from the time required to perceive the light, brake, and stop the vehicle, plus the assumed average speed into the intersection. Amber periods of 3–6 sec are typical.

- **Amber period:** The amber clearance period can be determined from the time required to perceive the light, brake, and stop the vehicle, plus the assumed average speed into the intersection. Amber periods of 3–6 sec are typical.

- **Green period:** The green period is the smaller of the sum of initial and vehicle periods and the maximum period.

- **All-red clearance period:** The all-red clearance period is a short period (i.e., 1 sec or 2 sec) that occurs after the amber period where all signals show red to allow for the full clearance of the intersection before the next green period begins.

### 24. PEDESTRIANS AND WALKWAYS

(Capacity analysis of pedestrian and bicycle movements in walkways is covered in HCM Chap. 16 as part of urban street analysis, in HCM Chap. 17 as part of intersections and roundabout analysis, and in HCM Chap. 28 as part of off-street facilities.)

The HCM recommends using an average walking speed of 4.0 ft/sec (1.22 m/s) and a 3 sec starting delay if elderly users constitute less than 20% of the walkway users. However, some people can walk as fast as 6 ft/sec (1.8 m/s), and 30–40% walk slower than 4 ft/sec (1.2 m/s). If more than 20% of walkway users are elderly, it may be appropriate to use a design speed of 3.3 ft/sec (1.0 m/s) and a 4–5 sec starting delay. Physically challenged individuals require additional consideration. An upgrade of 10% or greater reduces walking speed by 0.3 ft/sec (0.1 m/s).

The level of service (LOS) for pedestrians in walkways, sidewalks, and queuing areas is categorized in much the same way as for freeway and highway vehicles. Table 73.16 relates important parameters to the level of service. The primary criterion for determining pedestrian level of service is space (the inverse of density) per pedestrian. This, in turn, affects the speed at which pedestrians can walk. Mean speed and flow rate are supplementary criteria.

**Table 73.16 Pedestrian Level of Service on Walkways and Sidewalks**

<table>
<thead>
<tr>
<th>LOS</th>
<th>Average pedestrian density (ped/ft²)</th>
<th>Average pedestrian speed (ft/sec)</th>
<th>Flow rate (ped/min-ft)</th>
<th>Volume-capacity ratio (v/c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>&gt; 60</td>
<td>&gt; 4.25</td>
<td>≤ 5</td>
<td>≤ 0.21</td>
</tr>
<tr>
<td>B</td>
<td>&gt; 40–60</td>
<td>&gt; 4.17–4.25</td>
<td>&gt; 5–7</td>
<td>&gt; 0.21</td>
</tr>
<tr>
<td>C</td>
<td>&gt; 40–60</td>
<td>&gt; 4.00–4.17</td>
<td>&gt; 5–7</td>
<td>&gt; 0.21</td>
</tr>
<tr>
<td>D</td>
<td>&gt; 15–24</td>
<td>&gt; 3.75–4.00</td>
<td>&gt; 4.4–6.0</td>
<td>≥ 0.44</td>
</tr>
<tr>
<td>E</td>
<td>&gt; 15–24</td>
<td>&gt; 3.50–3.75</td>
<td>&gt; 3.75–4.0</td>
<td>≥ 0.44</td>
</tr>
<tr>
<td>F</td>
<td>≤ 8</td>
<td>≤ 3.50</td>
<td>≤ 3.50</td>
<td>variable</td>
</tr>
</tbody>
</table>

*This table does not apply to walkways with steep grades (e.g., >5%).*  
*Pedestrians per minute per foot width of walkway.*  
*Capacity = flow rate/23. LOS is based on average space per pedestrian.*  
*In cross-flow situations, the LOS-E threshold is 13 ft²/ped.*  

The peak pedestrian flow rate, \( v_{p,15} \), is the number of pedestrians passing a particular point during the 15 min peak period (ped/15 min). The effective walkway width, \( W_E \), is determined by subtracting any unusable width, including perceived “buffer zones” from adjacent features, from the total walkway width (ft or m). (Refer to HCM Exh. 24-11.) The average pedestrian unit flow rate (also known as unit width flow), \( v \), is the number of pedestrians passing a particular point per unit width of walkway (ped/min-ft).

\[
v = \frac{v_{p,15}}{15 W_E} \quad [HCM\ Eq. 23-3]
\]

**Pedestrian speed, \( S_p \), is the average pedestrian walking speed (ft/sec). Pedestrian density is the average number of pedestrians per unit area (ped/ft² or ped/m²).**
reciprocal of pedestrian density is pedestrian space, $A_p$ (ft$^2$/ped or m$^2$/ped), where $v_p$ is the pedestrian flow per unit width (ped/min-ft or ped/min-m).

$$A_p = \frac{S_p}{v_p} \quad [HCM \text{ Eq. 24-4}]$$

The maximum pedestrian capacity in walkways is 23 ped/min-ft, pedestrians per minute per foot of walkway width (75 ped/min-m). This occurs when the space is approximately 5–9 ft$^2$/ped (0.47–0.84 m$^2$/ped). Capacity drops significantly as space per pedestrian decreases, and movement effectively stops when space is reduced to 2–4 ft$^2$/ped (0.19–0.37 m$^2$/ped).

The HCM reports impeded flow starts at 530 ft$^2$/ped (49 m$^2$/ped), which is equivalent to 0.5 ped/min-ft (1.6 ped/min-m). These values are taken as the limits for LOS A. Also reported is that jammed flow in platoons starts at 11 ft$^2$/ped (1 m$^2$/ped), corresponding to 18 ped/min-ft (59 ped/min-m), which are used as the thresholds for LOS F. A platoon is a group of pedestrians walking together in a group.

### 25. CROSSWALKS

(Capacity analysis of pedestrians and bicycles at crosswalks is covered in HCM Chap. 18.)

Analysis of pedestrians in crosswalks is slightly different than in pure walkways. Walking is affected by signaling, turning vehicles, platooning, and interception of the platoon of pedestrians coming from the opposite side.

### 26. PARKING

The minimum parallel street parking stall width is commonly taken as 7 ft (2.1 m) in a residential area and may range from 8 ft to 11 ft (2.4 m to 3.3 m) in a commercial or industrial area. This width accommodates the vehicle and its separation from the curb. Stalls 7 ft (2.1 m) wide are substandard and should be limited to residential areas and attendant-parked lots. Widths larger than 9 ft (2.7 m) are appropriate in shopping areas where package loading is expected. If the width is specified between 10 ft and 12 ft (3.0 m and 3.7 m), the street parking corridor can be used for delivery trucks or subsequently converted to an extra traffic lane or bicycle path. The minimum length of a parallel street parking stall is 18 ft (5.4 m). In order to accommodate most cars, longer lengths between 20 ft and 26 ft (6.0 m and 7.8 m), may be used.

Figure 73.8 illustrates parallel street parking near an intersection as recommended by the AASHTO Green Book. The 20–28 ft (6–8.4 m) clearance from the last stall to the intersection is required to prevent vehicles from using the parking lane for right-turn movements.
The main advantage of SPUIs is that they require only a narrow right-of-way, thereby reducing land acquisition cost. The main disadvantage, as with all traffic bridges, is a high construction cost. There are additional geometric design features that require careful consideration, such as the elliptical left-hand turning path, pedestrian accommodations, and the difficulty in accommodating freeways approaching with high skew angles (e.g., more than 30°).

28. WEAVING AREAS

Analysis and design procedures for weaving areas are covered in HCM Chap. 12.

Weaving is the crossing of at least two traffic streams traveling in the same general direction along a length of highway without traffic control. In the freeway segment shown in Fig. 73.13, flows A–D and B–C cross the paths of other traveling vehicles, so these flows are the segment’s weaving movements. Flows A–C and B–D do not cross any other vehicles’ paths, so they are the segment’s nonweaving movements.

Weaving is an issue that must be considered in interchange selection, and interchanges without weaving are favored over interchanges with weaving. Weaving areas require increased lane-change maneuvers and result in increased traffic turbulence. Making a weaving segment longer allows for more time for lane changes. Under demand conditions, longer weaving segments increase capacity and decrease traffic density and turbulence.
The operating characteristics of a weaving segment are affected by the length, width, and configuration of the weaving segment. The *weaving segment length* is the distance between the merge and diverge segments. There are two measures of length used in the *HCM*, as shown in Fig. 73.14. The *short length*, \( L_S \), is the distance between the end points of any barrier markings (i.e., solid white lines) that discourage or prohibit lane changing. The *base length*, \( L_B \), is the distance between gore areas where the left edge of the ramp lane and the right edge of the freeway lane meet. Where no solid white lines are present, the two lengths are the same (i.e., \( L_S = L_B \)).

The *weaving segment width* is measured as the number of continuous lanes within the merging and diverging (i.e., entry and exit) gore areas. It is primarily controlled by the number of lanes on the entry and exit legs and the weaving segment configuration.

The *weaving segment configuration* refers to the relative placement and number of entry and exit lanes for a roadway section. Configuration is based on the number of required lane changes that must be performed by the

---

**Table:**

<table>
<thead>
<tr>
<th>type of intersecting facility</th>
<th>rural</th>
<th>suburban</th>
<th>urban</th>
</tr>
</thead>
<tbody>
<tr>
<td>local roads or streets</td>
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<td><img src="https://via.placeholder.com/150" alt="Diagram" /></td>
<td><img src="https://via.placeholder.com/150" alt="Diagram" /></td>
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<td>service interchanges</td>
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<td><img src="https://via.placeholder.com/150" alt="Diagram" /></td>
</tr>
<tr>
<td>collectors and arterials</td>
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<td>freeways systems interchanges</td>
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<td><img src="https://via.placeholder.com/150" alt="Diagram" /></td>
</tr>
</tbody>
</table>

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Previous editions of the *HCM* tied definitions of weaving segments to cloverleaf interchanges as most existing weaving segments were part of such interchanges. The *HCM* has replaced such definitions with a more general definition of length to reflect the fact that newer weaving segments occur in a variety of situations and designs.
two weaving flows in the section. The HCM identifies two types of weaving segment configurations: one-sided and two-sided weaving segments.

One-sided weaving segments have the entry and the exit from the weaving segment on the same side of the freeway. Figure 73.15 illustrates two types of one-sided weaving segments. Figure 73.15(a) illustrates a one-sided ramp weave where the on- and off-ramp are connected by a continuous auxiliary lane and where every weaving vehicle must make one lane change. Figure 73.15(b) illustrates a one-sided major weave where no continuous auxiliary lane is present and only the ramp to the freeway weaving segment requires a lane change. A major weave segment is a segment where three or more entry or exit legs have multiple lanes.

Two-sided weaving segments have an on-ramp on one side of the freeway closely followed by an off-ramp on the opposite side of the freeway. Figure 73.16(a) illustrates a two-sided weaving section with single-lane ramps where a one-lane, right-side on-ramp is closely followed by a one-lane, left-side off-ramp (or vice versa). It is the most common type of two-sided weave. Figure 73.16(b) illustrates a less common two-sided weave, where one of the ramps has multiple lanes. Typically, two-sided weaving segments require drivers to make three or more lane changes when entering or exiting the freeway. Though Fig. 73.16(a) only requires two lane changes, it still qualifies as a two-sided weaving segment because the entering and exiting vehicles create a weaving flow with the through-traffic movement.

29. TRAFFIC CALMING

Although traditional designs based on the HCM and the AASHTO Green Book have intended to maximize the flow of traffic, traffic calming features are introduced to slow the flow. Such features include traffic circles, narrow streets, curb extensions, and textured crosswalks. These features, which generally are contrary to traditional design guidelines in the AASHTO Green Book, are in response to a new urbanism that encourages consideration of historical, community, and aesthetic factors.

30. ECONOMIC EVALUATION

Road user costs include fuel, tires, oil, repairs, time, and accidents. Such costs affect design decisions where there are delays due to congestion, stops, turning, and so on. A reduction in road-user costs can be used to economically justify (i.e., as an economic warrant) interchanges and other features. Reductions in road-user costs are generally far greater than the increased cost of travel times that interchanges cause (compared to at-grade intersections).

Generally, public projects (such as interchanges) are justified using a benefit-cost ratio of annual benefits to annual capital costs. Annual benefits is the difference in road-user costs between the existing and improved conditions. Annual capital costs is the sum of interest and the amortization of the cost of the improvement. When staged construction is anticipated, incremental costs must be used to justify the construction of future subsequent stages.
18. FLEXIBLE PAVEMENT STRUCTURAL DESIGN METHODS

The goal of flexible pavement structural design methods is to specify the thicknesses of all structural layers in a pavement. Different methodologies are used for full-depth asphalt pavements, asphalt courses over an aggregate base, asphalt courses over emulsified base, and overlays. Aside from generic catalog methods, design methods can be divided into two types: state-of-the-practice empirical methods and state-of-the-art mechanistic and mechanistic-empirical (ME) methods. Like the choice of HMA mix design methods, the methodology used at the DOT level varies from state to state. All of the methods can be implemented manually or by computer.

Empirical methods, which include the 1993 AASHTO design procedure contained in the AASHTO Guide for Design of Pavement Structures (AASHTO GDPS), the 1998 Supplement containing a modified AASHTO design procedure, Asphalt Institute methods, and the Texas Modified Triaxial Design Method (not covered in this book), are based on the extrapolated performance of experimental test tracks.

ME methods build on multilayer elastic theory, finite element analysis, and simulation. They typically take into consideration reliability, climate, and life-cycle costs. Traffic is characterized by its spectrum, rather than in the form of a single (ESALs) number. Although ME methods have been implemented in Washington and Minnesota by WSDOT and MnDOT, respectively, for all practical purposes, ME design now refers only to the National Cooperative Highway Research Program (NCHRP) 1-37A method, as presented in the 2008 AASHTO Mechanistic-Empirical Pavement Design Guide (AASHTO MEPDG). ME methods require more training and situational awareness (environmental knowledge) than empirical methods and, unless implemented on a computer, tend to be dauntingly complex. Even then, computer “runs” can take hours.

Design of overlays of existing pavements can employ both empirical and mechanistic-empirical methods, as well as surface deflection methods using the results of falling weight deflectometer tests. These methods tend to use back-calculation of parameters obtained from in situ testing.

The design of an HMA pavement requires knowledge of climate, traffic, subgrade soils support, and drainage. Stiffness of the asphalt layer varies with temperature, and unbound layers (aggregate and subgrade) are affected by freeze-thaw cycles. Climate is characterized by the mean annual air temperature, MAAT, of the design area. MAATs of 45°F (7°C) or less require attention to frost effects; at 60°F (16°C), frost effects are considered possible; and at 75°F (24°C), frost effects can be neglected. In empirical methods, the traffic is characterized by the numbers and weights of truck and bus axle loads expected during a given period of time, specified in ESALs. ESALs are calculated by multiplying the number of vehicles in each weight class by the appropriate truck factor and summing the products. Consideration must be given to subsurface drainage (e.g., installation of underdrains and/or interceptor drains) where high water tables occur or where water may accumulate in low areas. Good surface drainage, obtained through proper crown design, is also essential.

19. TRAFFIC

(The information included in this section is consistent with AASHTO GDPS, 1993.)

The AASHTO pavement design method requires that all traffic be converted into equivalent single-axle loads (ESALs). This is the number of 18,000 lbf single axles (with dual tires) on pavements of specified strength that would produce the same amount of traffic damage over the design life of the pavement.

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Table 76.6 Minimum VMA Requirements and VFA Range Requirements

<table>
<thead>
<tr>
<th>20 yr traffic loading (in millions of ESALs)</th>
<th>minimum VMA (%)</th>
<th>VFA range (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.5 mm (0.375 in)</td>
<td>15.0</td>
<td>70–80</td>
</tr>
<tr>
<td>12.5 mm (0.5 in)</td>
<td>14.0</td>
<td>65–78</td>
</tr>
<tr>
<td>19.0 mm (0.75 in)</td>
<td>13.0</td>
<td>65–75</td>
</tr>
<tr>
<td>25.0 mm (1 in)</td>
<td>12.0</td>
<td></td>
</tr>
<tr>
<td>37.5 mm (1.5 in)</td>
<td>11.0</td>
<td></td>
</tr>
</tbody>
</table>

(Multiply in by 25.4 to obtain mm.)


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5Mechanistic methods are based on familiar mechanics (strengths) of materials concepts that relate wheel loading to pavement stresses, strains, and deflections according to the material properties. The term “mechanistic-empirical” acknowledges that the theory is calibrated and corrected according to observed performance.